Older Driver Highway Design Handbook

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Research and Development
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FOREWORD

The proportion of the driving population over 65 is growing significantly. Older motorists can be expected to have problems driving given the known changes in their perceptual, cognitive, and psychomotor performances, presenting many challenges to transportation engineers, who must ensure system safety while increasing operational efficiency.

This *Older Driver Highway Design Handbook* provides practitioners with a practical information source that links older road user characteristics to highway design, operational, and traffic engineering recommendations by addressing specific roadway features. This handbook supplements existing standards and guidelines in the areas of highway geometry, operations, and traffic control devices.

The information in this handbook should be of interest to highway designers, traffic engineers, and highway safety specialists involved in the design and operation of highway facilities. In addition, this handbook will be of interest to researchers concerned with issues of older road user safety and mobility.

Copies of this report can be obtained through the FHWA Research and Technology Report Center, 9701 Philadelphia Court, Unit Q, Lanham, Maryland 20706, telephone: (301) 577-0818, fax: (301) 577-1421, or the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161, telephone: (703) 487-4650, fax: (703) 321-8547.

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Director
Office of Safety and Traffic Operations
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16. Abstract

This project included literature reviews and research syntheses, using meta-analytic techniques where appropriate, in the areas of age-related (diminished) functional capabilities, and human factors and highway safety. A User-Requirements Analysis to gauge the needs of highway design and engineering professionals for guidance in accommodating older drivers was also performed. Together, these efforts supported development of three research products: (1) an applications-oriented *Older Driver Highway Design Handbook* intended to supplement standard design manuals for practitioners; *(2)* an *Older Driver Research Synthesis* (FHWA-RD-97-094), oriented toward human factors professionals and researchers; and *(3)* a *Human Factors and Highway Safety Synthesis* (FHWA-RD-97-095) capturing major findings and trends in studies of driver use of (and difficulties with) a wide range of highway elements.

Additional project activities included review of this product, the *Older Driver Highway Design Handbook*, by a panel of 22 state and local practitioners, who applied draft recommendations to real-world highway engineering problems and suggested needed changes to improve the accuracy, presentation, and accessibility of this information. A subsequent revision of this document was performed to incorporate practitioners' review comments.

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Safety, Highway Design, Highway Operations, Driver Age, Driver Performance, Human Factors, Vision, Attention, Perception, Cognition, Memory, Physical Ability, Risk Perception, Hazard Perception.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm'
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.636	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.765	liters	L
ft^3	cubic feet	0.026	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Vdumes greater than 1000 I shall be shown in m^3.				
		MASS		
oz	ounces	26.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (oi "metric ton')	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.6	Cekius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.69	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.26	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
'ha	hectares	2.47	acres	ac
km^2	square kilometers	0.366	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.397	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton')	1.103	short tons (2090 lb)	T
		TEMPERATURE (exact)		
°C	Cekius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kifopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E360.

(Revised September 1993)

ACKNOWLEDGMENTS

At the outset of ***Handbook*** development, it was recognized that many traffic engineers have already identified for themselves one or more priority issues where age-related declines in driver performance capability defme the need for modifications or enhancements of current practice. This ***Handbook*** was therefore built upon the results of a ***user*** requirements ***analysis,*** in which 94 practitioners from 5 national committees provided detailed feedback indicating how the highway design and engineering communitycould most effectively use older driver data in design, operational, and safety decisions. A two-stage review process incorporating a lengthy and detailed survey was undertaken to yield consensus regarding the most useful contents and format for the ***Handbook.*** Participating committees in the user requirements analysis included the American Association of State and Highway Transportation Officials (AASHTO) Subcommittee on Design; the National Committee on Uniform Traffic Control Devices; the AASHTO Standing Committee on Highway Traffic Safety; the Transportation Research Board (TRB) Committee on Geometric Design (A2A02); and the TRB Committee on the Operational Effects of Geometrics (A3A08). The conscientious response by the practitioners contacted through these committees is gratefully acknowledged.

As ***Handbook*** development proceeded, a more rigorous requirement for review and criticism of draft recommendations and supporting materials was defined. Specifically, a need was identified to determine the utility of the ***Handbook*** for its intended users-practicing engineers at the State and local levels. The critical review by a panel of individuals who are presently engaged in engineering practice or have recent past experience as practicing engineers was solicited, with the active support of three key committee chairmen: Mr. Thomas Wame, AASHTO Subcommittee on Design; Mr. Richard Weaver, AASHTO Subcommittee on Traffic Engineering; and Mr. Ken Kobetsky, National Committee on Uniform Traffic Control Devices.

Members of the review panel, which represented to a roughly equal degree the design and operational sides of the highway engineering community, were asked to: (1) apply draft recommendations for one or more design elements from ***the Handbook*** in case studies involving real-world engineering problems where older driver performance has been (or could be) an important variable; (2) provide structured responses using rating scales to identify needed changes in the information presented in the ***Handbook***; and (3) provide open-ended responses and edits of ***Handbook*** material as deemed necessary to improve its accuracy, accessibility, or presentation. A frank discussion of the relevance of each recommendation reviewed by panel members was requested, in the sense of whether it contributed to an improved solution to the problem under study and would be consulted freely for applications apart from this research, or whether the practitioner deemed it irrelevant or confusing and would not be likely to consult this reference in the future. It is only as a result of the thoughtful responses of the individuals listed on the following page that revision of the earlier draft into a completed document was accomplished.

Special thanks are extended to the practitioners who provided ***critical*** review of ***Handbook*** materials. The following persons provided thoughtful comments-expressing both agreement and disagreement-on a draft version of this document:

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PREFACE

The increasing numbers and percentages of older drivers using the Nation's highways in the decades ahead will pose many challenges to transportation engineers, who must ensure system safety while increasing operational efficiency. The 65 and older age group, which numbered 33.5 million in the United States in 1995, will grow to more than 36 million by 2005 and will exceed 50 million by 2020, accounting for roughly one-fifth of the population of driving age in this country. In effect, if design is controlled by even 85th percentile performance requirements, the "design driver" of the early 21st century will be an individual over the age of 65.

There are important consequences of the changing demographics in our driving population. Traffic volumes will increase, problems with congestion will become more widespread, and the demands on drivers will grow significantly beyond present-day operating conditions. At the same time, a steadily increasing proportion of drivers will experience declining vision; slowed decisionmaking and reaction times; exaggerated difficulty in dividing attention between rapidly shifting sources of potential conflicts and other traffic information; and reductions in strength, flexibility, and overall fitness.

A premise for development of ***the Older Driver Highway Design Handbook*** is that practitioners, while generally aware of the current number and projected increases in the number of older drivers, do not presently have access to any practical information source linking the characteristics of these highway users to design, operational, and traffic engineering recommendations keyed to specific roadway features. This ***Handbook*** has accordingly been developed to supplement existing standards and guidelines in the areas of highway geometry, operations, and traffic control devices.

The specific roadway features singled out for attention in this ***Handbook*** represent four broad site types identified either directly or indirectly in recent accident analyses as most problematic for older drivers. A top priority is at-grade ***intersections,*** reflecting older drivers' most serious accident problem area as documented in recent analyses (Council and Zegeer, 1992; Staplin and Lyles, 1991; Stamatiadis, Taylor, and McKelvey , 1991). Next, older driver difficulties with merginglweaving and lane changing operations focus attention on ***interchanges (grade separation).*** Finally, ***roadway curvature*** and ***passing zones*** plus highway ***construction/work*** zones are included for two reasons: (1) heightened tracking (steering) demands may increase the driver's workload, and (2) there is an increased potential for unexpected events requiring a swift driver response.

These classes of highway features define the primary organizing principle for the main body of the ***Handbook.*** Recommendations are presented initially in a brief section, followed by a more lengthy section presenting the Rationale and Supporting Evidence. Within each of these two major ***Handbook*** sections, material is organ&d in terms of four subsections, corresponding to the classes of highway features noted above. Then, for each class of highway feature, ***Handbook*** materials are organized according to a unique set of geometric, operational, and traffic control design elements. The ***Handbook*** concludes with an integrated glossary providing definitions of selected terms, including acronyms and abbreviations; a reference list; and an index containing terms that provide reliable guidance to help locate ***Handbook*** entries pertaining to a particular design element.

The recommendations in ***this Handbook*** are based on supporting evidence drawn from a selected set of research findings. The results of field studies employing older drivers were always given precedence, followed by laboratory simulations or modeling efforts where both age and some aspect of highway design, operations, or traffic control were included as study variables. More general findings on the effects of aging, independent of driver performance research per se, may also be cited, but only where there is an indisputable logic extending a given finding to the highway context. A broader discussion of issues related to aging and driving can be found in the Transportation Research Board's ***Special Report 218*** (1988).

It is important to emphasize that ***Handbook*** recommendations, as well as the evidence cited to support them, relate to demonstrated performance deficits of ***normally aging*** drivers. Thus, diminished driver capabilities that result from the onset of Alzheimer's disease and related dementias, which are believed to afflict over 10 percent of those age 65 and older and nearly 50 percent of those age 85 and older, are not the current focus.

Finally, the recommendations presented in this ***Handbook*** do not constitute a new standard of required practice for the included highway design elements. Questions related to when and where to apply each ***Handbook*** recommendation remain at the discretion of the practitioner. This document may be useful as a "problem solver" at older driver accident sites, or it may be applied preemptively to enhance safety wherever there are large numbers of older drivers in the traffic stream in a given jurisdiction. As a practical matter, it is recognized that the application of ***Handbook*** recommendations may be limited to the design of new facilities and to planned highway reconstruction projects. Furthermore, the recommendations contained herein seek to avoid "optimum" solutions that may be unattainable using current materials or practices or that will result in situations where extreme costs are incurred for small anticipated gains in system safety. Ultimately, the contents of ***this Handbook*** are intended to provide guidance which-based on the current state-of-the-knowledge of the special needs of normally aging seniors-can be expected to significantly enhance the safety and ease of use of the highway system for older drivers in particular, and for the driving population as a whole.

Loren Staplin, Ph.D.
Kathy H. Lococo
Stanley R. Byington

October 1997

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ABBREVIATIONS AND ACRONYMS

AAAFTS . . . . American Automobile Association Foundation for Traffic Safety

AADT . . . . annual average daily traffic

AASHTO . . . . American Association of State and Highway Transportation Officials

ATSSA . . . . American Traffic Safety Services Association

cd . . . . candela

CIE . . . . Commission Intemationale de **l'Eclairage**

CIL . . . . complete interchange lighting

CSSB . . . . concrete safety shaped barrier

DS . . . . diverge steering

DSD . . . . decision sight distance

FARS . . . . Fatal Accident Reporting System

FHWA . . . . Federal Highway Administration

fL . . . . footlambert

GSA . . . . gap search and acceptance

hfc . . . . horizontal footcandle

IA . . . . initial acceleration

IIHS . . . . Insurance Institute for Highway Safety

ISD . . . . intersection sight distance

ISBL . . . . in-service brightness level

ISTEA . . . . Intermodal Surface Transportation Efficiency Act

ITE . . . . Institute of Transportation Engineers

LI . . . . legibility index

MOE . . . . measure of effectiveness

ABBREVIATIONS AND ACRONYMS (Continued)

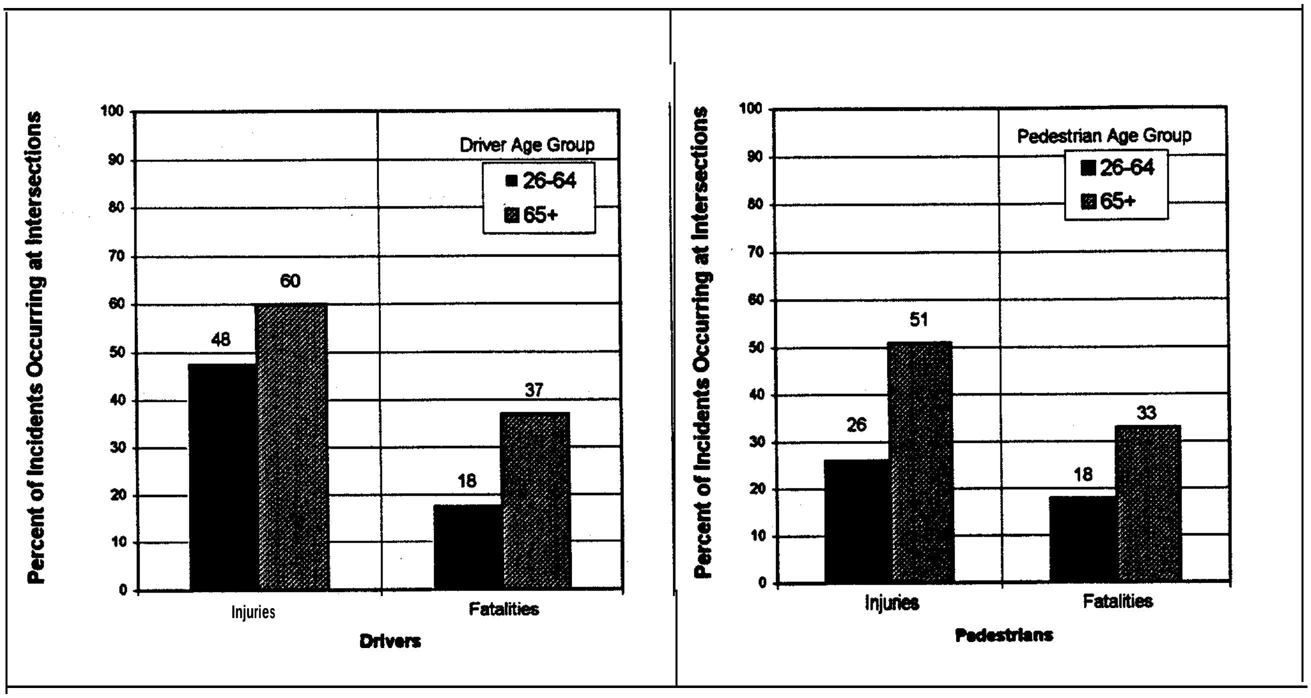
MRVD	minimum required visibility distance
MSC	merge steering control
MUTCD	*Manual on UniformTraffic Control Devicesfor Streets and Highways*
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTOR	no turn on red
NTSB	National Transportation Safety Board
PIL	partial interchange lighting
PMD	post-mounted delineator
PRT	perception-reaction time
RT	reactiontime
RPM	raised pavement markers
RTOR	right turn on red
SC	steering control
SCL	speed-changelane
SSD	stopping sight distance
STV	small target visibility
TCD	traffic control device
TRB	Transportation Research Board
TVA	transient visual adaptation
UFOV	useful field of view
VC	visual clear
VMS	variable message sign

RECOMMENDATIONS

I. INTERSECTIONS (AT-GRADE)

Background and Scope of *Handbook* Recommendations

The single greatest concern in accommodating older road users, both drivers and pedestrians, is the ability of these persons to safely maneuver through intersections. The findings of one widely cited analysis of nationwide accident data (Hauer, 1988), illustrated below, reveal the relationship between injuries and fatalities at intersections during the period 1983-1985 in the United States, as a function of age and road user type (driver or pedestrian).



For drivers 80 years and older, ***more than half* of** fatal accidents occur at intersections, compared with 24 percent or less for drivers up to 50 years of age (Insurance Institute for Highway *Safety,* 1993). These findings reinforce a long-standing recognition that driving situations involving complex speed-distance judgments under time constraints-the typical scenario for intersection operations-are more problematic for older drivers and pedestrians than for their younger counterparts (Waller, House, and Stewart, 1977). Other studies within the large body of evidence showing dramatic increases in intersection accident involvements as driver age increases have revealed detailed patterns of data associating specific accident types and vehicle movements with particular age groups, and in some cases have linked such patterns to the driving task demands in a given maneuver situation (see Campbell, 1993; Council and Zegeer, 1992; Staplin and Lyles, 1991).

Another approach to characterizing older driver problems at intersections was employed by Brainin (1980), who used in-car observations of driving behavior with 17 drivers ages 25-44, 81 drivers ages 60-69, and 18 drivers age 70 and older, on a standardized test route. The two older age groups showed more difficulty making right and left turns at intersections and negotiating traffic signals. The left-turn problems resulted from a lack of sufficient caution and poor positioning on the road during the turn. Right-turn difficulties were primarily a result of failing to signal. Errors demonstrated at STOP signs included failing to make complete stops, poor vehicle positioning at STOP signs, and jerky and abrupt stops. Errors demonstrated at traffic signals included stops that were either jerky and abrupt, failure to stop when required, and failure to show sufficient caution during the intersection approach.

Complementing accident analyses and observational studies with subjective reports of intersection driving difficulties, a statewide survey of 664 senior drivers by Benekohal, Resende, Shim, Michaels, and Weeks (1992) found that the following activities become more difficult for drivers as they grow older (with proportion of drivers responding in parentheses):

- Reading street signs in town (27 percent).
- Driving across an intersection (21 percent).
- Finding the beginning of a left-turn lane at an intersection (20 percent).
- Making a left turn at an intersection (19 percent).
- Following pavement markings (17 percent).
- Responding to traffic signals (12 percent).

Benekohal et al. (1992) also found that the following highway features become more important to drivers as they age (with proportion of drivers responding in parentheses):

- Lighting at intersections (62 percent).
- Pavement markings at intersections (57 percent).
- Number of left-turn lanes at an intersection (55 percent).
- Width of travel lanes (5 1 percent).
- Concrete lane guides (raised channelization) for turns at intersections (47 percent).
- Size of traffic signals at intersections (42 percent).

Comparisons of responses from drivers ages 66-68 versus those age 77 and older showed that the older group had more difficulty following pavement markings, finding the beginning of the left-turn lane, and driving across intersections. Similarly, the level of difficulty for reading street signs and making left turns at intersections increased with increasing senior driver age. Turning left at intersections was perceived as a complex driving task. This was made more difficult when raised channelization providing visual cues was absent, and only pavement markings designated which were through lanes versus turning lanes ahead. For the oldest age group, pavement markings at intersections were the most important item, followed by the number of left-turn lanes, concrete guides, and intersection lighting. A study of older road users completed in 1996 provides evidence that the single most challenging aspect of intersection negotiation for this group is performing left turns during the permitted (green ball) signal phase (Staplin, Harkey, Lococo, and Tarawneh, 1997).

During focus group discussions conducted by Benekohal et al. (1992), older drivers reported that intersections with too many islands are confusing, that raised curbs that are unpainted are difficult to see, and that textured pavements (rumble strips) are of value as a warning of upcoming raised medians, approaches to (hidden or flashing red) signals, and the roadway edge/shoulder lane boundary. Regarding traffic signals, study subjects indicated a clear preference to turn left on a protected arrow phase, rather than making "permitted phase" turns. When turning during a permitted phase (green ball) signal operation, they reported waiting for a large gap before making a turn, which frustrates drivers in back of them and causes the drivers behind to go around them or blow their horns. A general finding here was the need for more time to react.

Additional insight into the problems older drivers experience at intersections was provided by focus group responses from 81 older drivers in the Staplin et al. study (1997). The most commonly reported problems are listed below:

- Difficulty in turning head at skewed (non-go-degree) angles to view intersecting traffic.
- Difficulty in smoothly performing turning movements at tight comers.
- Hitting raised concrete barriers such as channelizing islands in the rain and at night due to poor visibility.
- Finding oneself positioned in the wrong lane-especially a "turn only" lane-during an intersection approach, due to poor visibility (maintenance) of pavement markings or the obstruction of roadside signs designed to inform drivers of intersection traffic patterns.
- Difficulty at the end of an auxiliary (right)-turn lane in seeing potential conflicts well and quickly enough to smoothly merge with adjacent-lane traffic.
- Merging with adjacent-lane traffic after crossing an intersection, when a lane drop occurs near the intersection (e.g., when two lanes merge into one lane within 150 m [500 ft] after crossing the intersection).

Although these problems are by no means unique to older drivers, the various functional deficits associated with aging result in exaggerated levels of difficulty for this user group.

Finally, the analysis by Council and Zegeer (1992) included an examination of pedestrian accidents and the collision types in which older pedestrians were overinvolved. The results showed older pedestrians to be overrepresented in both right- and left-turn accidents. The young-elderly (ages 65-74) were most likely to be struck by a vehicle turning right, whereas the old-elderly (age 75 and older) were more likely to be struck by a left-turning vehicle.

This section will provide recommendations to enhance the performance of diminished-capacity drivers as they approach and travel through intersections, for 16 different design elements: A. intersecting angle (skew); B. receiving lane (throat) width for turning operations; C. channelization; D. intersection sight distance (sight triangle); E. opposite (single) left-turn lane geometry, signing, and delineation; F. edge treatments/delineation of curbs, medians, and obstacles; G. curb radius; H. traffic control for left-turn movements at signalized intersections; I. traffic control for right-turn/right-turn-on-red (RTOR) movements at signalized intersections; J. street-name signage; K. one-way/wrong-way signage; L. stop- and yield-controlled intersection signage; M. devices for lane assignment on intersection approach; N. traffic signal performance issues; 0. fixed lighting installations; and P. pedestrian control devices.

The *Handbook* recommendations that follow are supported by material presented later in the "Rationale and Supporting Evidence" section under the "Intersections (At-Grade)" heading.

Recommendations by Design Element

A. *Design Element: Intersecting Angle (Skew)*

(1) In the design of new facilities where right-of-way is not restricted, all intersecting roadways should meet at a 90-degree angle.

(2) In the design of new facilities or redesign of existing facilities where right-of-way is restricted, intersecting roadways should meet at an angle of not less than 75 degrees.

The rationale and supporting evidence for these recommendations can be found beginning on page 42 of this Handbook.

B. *Design Element: Receiving Lane (Throat) Width for Turning Operations*

(1) A minimum receiving lane width of 3.6 m (12 ft) is recommended, accompanied, wherever practical, by a shoulder of 1.2 m (4 ft) minimum width.

The rationale and supporting evidence for this recommendation can be found beginning on page 44 of this Handbook.

C. *Design Element: Channelization*

(1) At intersections with a high volume of pedestrians, it is recommended that right-turn channelization *not* be implemented without the provision of an adjacent pedestrian refuge island conforming to MUTCD (Federal Highway Administration, 1988) and AASHTO (1994) specifications.

(2) If right-turn channelization is present at an intersection, an acceleration lane providing for the acceleration characteristics of passenger cars as delineated in AASHTO specifications (1994) is recommended.

C. *Design Element:* ***Channelization*** (Continued)

(3) Raised channelization (sloping curbed medians) is recommended over painted channelization for left- and right-turn lane treatments at intersections, with island curb sides and curb surfaces treated with reflectorized paint and maintained at a minimum luminance contrast level of 3.0 or higher under low beam (passenger vehicle) headlight illumination.

The rationale and supporting evidence for these recommendations can be found beginning on page 46 of this Handbook.

D. Design Element: Intersection Sight Distance (Sight Triangle)

(1) For Cases I through IV, as described below, it is recommended that perception-reaction time (PRT) for intersection sight distance (ISD) be no less than 2.5 s to accommodate the slower decision times exhibited by, and the larger gap sizes desired, by older drivers.

Case I: No Control
Case II: Yield Control
Case IIIA: Stop Control—Crossing
Case IIIB: Stop Control—Left Turn
Case IIIC: Stop Control—Right Turn
Case IV: Signal Control

(2) For ISD Case V (Stop Control—Vehicle Turning Left From Major Highway), unrestricted sight distances and corresponding left-turn lane offsets are recommended whenever possible in the design of opposite left-turn lanes at intersections.

(2a) At intersections where there are large percentages of left-turning trucks, the offsets required to provide unrestricted sight distance for opposing left-turn trucks should be used.

D. Design Element: Intersection Sight Distance (Sight Triangle) (Continued)

(2b) Where the provision of unrestricted sight distance is not feasible, ISD values for left-turning traffic that must yield to opposing traffic on the major roadway (ISD Case V) should be computed using the modified AASHTO model, as follows:

$ISD = 1.47\ V\ (J + t_a)$	**English**
$ISD = 0.278\ V\ (J + t_a)$	**Metric**

where:

ISD = intersection sight distance (feet for English equation; meters for metric equation).

V = major roadway operating speed (mi/h for English equation; km/h for metric equation).

J = time to search for oncoming vehicles, to perceive that there is sufficient time to make the left turn, and to shift gears, if necessary, prior to starting (modified to 2.5 s).

t_a = time required to accelerate and traverse the distance to clear traffic in the approaching lane(s); obtained from figure IX-33 in the AASHTO Green Book.

The rationale and supporting evidence for these recommendations can be found beginning on page 50 of this Handbook.

E. Design Element: Opposite (Single) Left-Turn Lane Geometry, Signing, and Delineation

(1) Unrestricted sight distance (achieved through positive offset of opposite left-turn lanes) is recommended whenever possible, for new or reconstructed facilities. This will provide a margin of safety for older drivers who, as a group, do not position themselves within the intersection before initiating a left turn.

(2) At intersections where engineering judgment indicates a high probability of heavy trucks as the opposing turn vehicles during normal operations, the offsets required to provide unrestricted sight distance for opposing left-turn trucks should be used, for new or reconstructed facilities.

E. Design Element: Opposite (Single) Left-Turn Lane Geometry, Signing, and Delineation (Continued)

(3) Where the provision of unrestricted sight distance is not feasible, ISD values for left-turning traffic that must yield to opposing traffic on the major roadway (ISD Case V) should be computed using the modified AASHTO model, as follows:

$ISD = 1.47\ V\ (J + t_a)$	**English**
$ISD = 0.278\ V\ (J + t_a)$	**Metric**

where:

- **ISD = intersection sight distance (feet for English equation; meters for metric equation).**
- **V = major roadway operating speed (mi/h for English equation; km/h for metric equation).**
- **J = time to search for oncoming vehicles, to perceive that there is sufficient time to make the left turn, and to shift gears, if necessary, prior to starting (modified to 2.5 s).**
- **t_a = time required to accelerate and traverse the distance to clear traffic in the approaching lane(s); obtained from Figure IX-33 in the AASHTO Green Book (1994).**

(4) At intersections where the left-turn lane treatment results in channelized offset left-turn lanes (e.g., a parallel or tapered left-turn lane between two medians) the following countermeasures are recommended to reduce the potential for wrong-way maneuvers by drivers turning left from a stop-controlled, intersecting minor roadway:

(4a) In the implementation of (advance) DIVIDED HIGHWAY CROSSING signs, and WRONG WAY, DO NOT ENTER, and ONE WAY signs at the intersection, as per MUTCD (Federal Highway Administration, 1988) specifications, sign sizes larger than MUTCD standard sizes (e.g., MUTCD expressway size for DO NOT ENTER [900 x 900 mm] and MUTCD special size for WRONG WAY [1050 x 750 mm]) are recommended, as is high-intensity sheeting.

(4b) Lane-use arrows for channelized left-turn lanes are recommended, and reflectorized treatments should be used wherever practical; otherwise, white painted pavement markings should be used.

E. Design Element: Opposite (Single) Left-Turn Lane Geometry, Signing, and Delineation (Continued)

(4c) Pavement markings which scribe a path through the turn are recommended to reduce the likelihood for the wrong-way movement.

(4d) The use of a white stop bar 600 mm (24 in) in width is recommended at the end of the channelized left-turn lane as a countermeasure to aid in preventing a potential wrong-way movement.

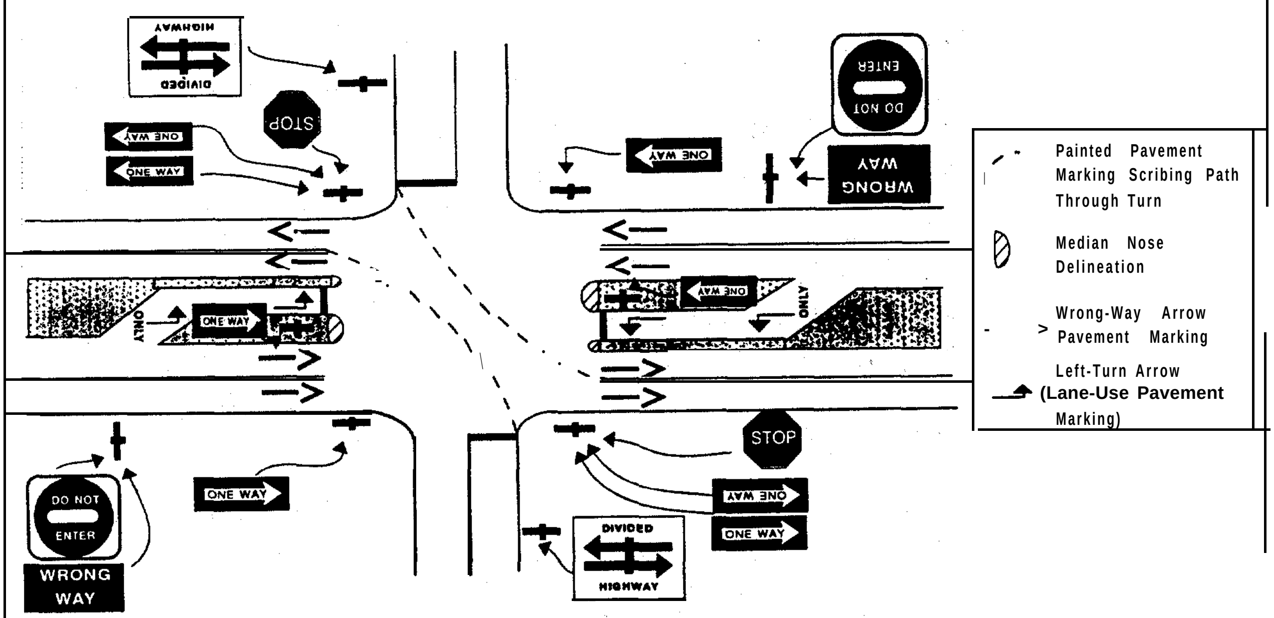
(4e) Placement of 7-m (23.5-ft) wrong-way arrows in the through lanes is recommended for wrong-way traffic control at locations determined to have a special need, as specified in the MUTCD, section 2E-40.

(4f) Delineation of median noses using reflectorized paint and other treatments to increase their visibility and improve driver understanding of the intersection design and function is recommended.

The diagram presented on the facing page illustrates the countermeasures as described above in *Handbook* Recommendations E(4a)-(4f).

The rationale and supporting evidence for these recommendations can be found beginning on page 61 of this Handbook.

E. Design Element: Opposite (Single) Left-Turn Lane Geometry, Signing, and Delineation (Continued)



Recommended signing and delineation treatments for intersections with medians 9 m (30 ft) wide or wider, and medians with channelized left-turn lanes, to reduce the potential for wrong-way movements for drivers turning left from the minor roadway.

F. Design Element: Edge Treatments/Delineation of Curbs, Medians, and Obstacles

(1) A minimum in-service contrast level of 2.0 is recommended between the painted edge of the roadway and the road surface for intersections with overhead lighting, where:

$$luminance\ (L)\ contrast = \frac{L_{stripe} - L_{pavement}}{L_{pavement}}$$

(2) A minimum in-service contrast level of 3.0 is recommended between the painted edge of the roadway and the road surface for intersections without overhead lighting.

(3) It is recommended that all curbs at intersections (including median islands and other raised channelization) be delineated on their vertical face and at least a portion of the top surface, in addition to the provision of a painted edgeline on the road surface.

(4) It is recommended that a "preview" of vertical surfaces be provided using cross-hatched pavement markings, as specified in the MUTCD (Federal Highway Administration, 1988), section 3B-13.

The rationale and supporting evidence for these recommendations can be found beginning on page 67 of this Handbook.

G. Design Element: Curb Radius

(1) Except where precluded by high volumes of heavy vehicles, a corner curb radius of 9 m (30 ft) is recommended as a tradeoff to (a) facilitate vehicle turning movements, (b) moderate the speed of turning vehicles, and (c) avoid unnecessary lengthening of pedestrian crossing distances.

The rationale and supporting evidence for these recommendations can be found beginning on page 70 of this Handbook.

H. Design Element: Traffic Control for Left-Turn Movements at Signalized Intersections

(1) The use of protected-only operations is recommended, except when, based on engineering judgment, an unacceptable reduction in capacity will result.

(2) To reduce confusion during an intersection approach, the use of a separate signal to control movements in each lane of traffic is recommended.

(3) Consistent use of a common sign throughout the United States advising drivers of the correct response to a steady green ball during protected-permitted operations (R10-12, "LEFT TURN YIELD ON GREEN ●") is recommended, with overhead placement preferred at the intersection.

(4) A leading protected left-turn phase is recommended wherever protected left-turn signal operation is implemented.

(5) To reduce confusion about the meaning of the red arrow indication, it is recommended that the steady green arrow for protected-only left-turn operations terminate to a yellow arrow, then a steady red ball.

(6) The use of redundant upstream signing (R10-12) is recommended to advise left-turning drivers of permitted signal operation. It is also recommended that the signing afford at least a 3-s preview (at operating speeds in the left-turn lane) before the intersection, using either overhead or median sign placement.

H. Design Element=Traffic Control for Left-Turn Movements at Signalized Intersections (Continued)

(7) Where the required (minimum) sight distance as calculated using a modified AASHTO intersection sight distance (ISD) model with a 2.5-s perception-reaction time (PRT) (see *Handbook* Chapter I, Design Element E) is not practical to achieve through geometric redesign/reconstruction, the following operational changes are recommended:

(7a) If programmable signal control capability exists, restrict permitted left-turn operations to low-volume (off-peak) conditions.

(7b) Where a pattern of permitted left-turn accidents occurs, eliminate permitted left turns and implement protected-only left-turn operations.

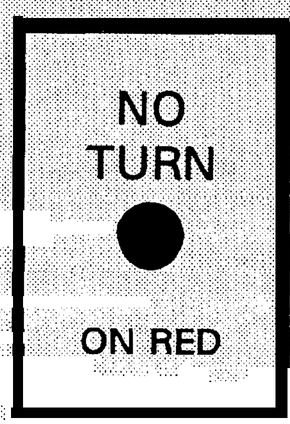
The rationale and supporting evidence for these recommendations can be found beginning on page 74 of this Handbook.

I. Design Element: Traffic Control for Right-Tum/RTOR Movements at Signalized Intersections

(1) To reduce confusion with the meaning of the (right-turn) red arrow, it is recommended that a steady red ball be used at signalized intersections where a right turn is prohibited, supplemented by the NO TURN ON RED sign depicted in Recommendation 3 below.

(2) The prohibition of right turn on red (RTOR) at skewed intersections (angle less than 75 degrees or greater than 105 degrees) is recommended.

(3) The signing of prohibited RTOR movements using the novel design (as shown) is recommended, with sign placement on the overhead mast arm and on the opposite corner of the intersection.



I. Design Element: ***Traffic Control*** for ***Right-Turn/RTOR*** Movements ***at Signalized Intersections*** (Continued)

(4) Where RTOR is permitted and a pedestrian crosswalk is delineated on the intersecting roadway, the posting of signs with the legend TURNING TRAFFIC MUST YIELD TO PEDESTRIANS is recommended, in an overhead or roadside location that is easily visible to the motorist prior to initiating the turning maneuver.

The rationale and supporting evidence for these recommendations can be found beginning on page 82 of this Handbook.

J. Design Element: Street-Name Signage

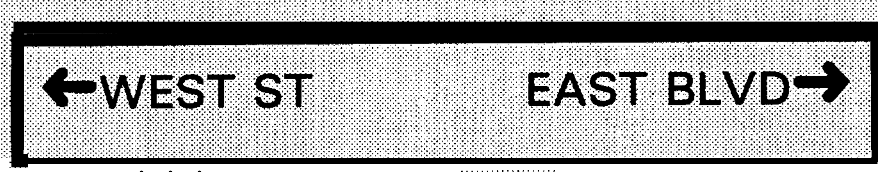
(1) To accommodate the reduction in visual acuity associated with increasing age, a [redacted] letter height of 150 mm (6 in) is recommended for use on post-mounted street-name signs (D3).

(2) The use of overhead-mounted street-name signs with minimum letter heights of 200 mm (8 in) is recommended at major intersections.

(3) Wherever an advance intersection warning sign is erected (e.g., W2-1, W2-2, W2-3, W2-4), it is recommended that it be accompanied by a supplemental street-name sign.

(4) The use of redundant street-name signing for major intersections is recommended, with an advance street-name sign placed upstream of the intersection at a midblock location, and an overhead-mounted street-name sign posted at the intersection. Wherever practical, the midblock sign should be mounted overhead.

(5) When different street names are used for different directions of travel on a crossroad, the names should be separated and accompanied by directional arrows on both midblock and intersection street-name signs, as shown below:

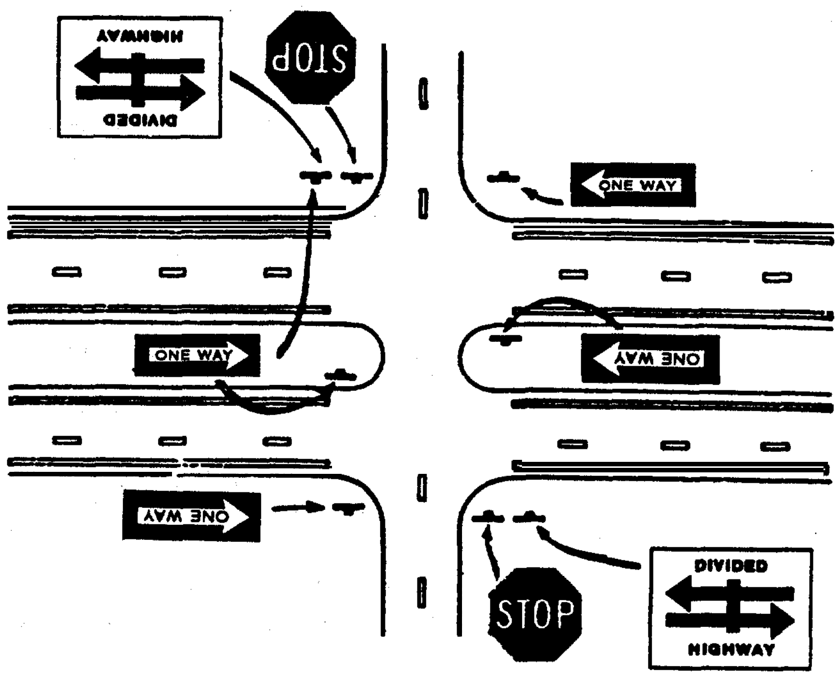


The rationale and supporting evidence for these recommendations can be found beginning on page 87 of this Handbook.

K. Design Element: One-Way/Wrong-Way Signage

(1) It is recommended that approaches to divided highways be consistently signed; use of the DIVIDED HIGHWAY CROSSING sign (R6-3) is the recommended current practice, but this sign may be replaced or supplemented with new treatments that are demonstrated through research to provide improved comprehensibility to motorists.

(2) For divided highways with medians of 9 m (30 ft) and under, the use of four ONE WAY signs is recommended, as shown in the configuration diagrammed below.



Recommended signing configuration for medians less than or equal to 9 m (30 ft).

(3) For medians over 9 m (30 ft), the use of eight ONE WAY signs is recommended, as diagrammed in Recommendation 4 of Design Element E.

K. Design Element: One-Way/Wrong-Way Signuge (Continued)

(4) For T-intersections, the use of a near-right side ONE WAY sign and a far side ONE WAY sign is recommended; the preferred placement for the far side sign is opposite the extended centerline of the approach leg as shown in MUTCD figure 2-4 (Federal Highway Administration, 1988). Where the preferred far side location is not feasible because of blockage, distracting far side land use, or an excessively wide approach leg, etc., engineering judgment should be applied to select the most conspicuous alternate location for a driver who has not yet initiated the wrong-way turning maneuver.

(5) For four-way intersections (i.e., the intersection of a one-way street with a two-way street), ONE WAY signs placed at the near right/far left locations are recommended, regardless of whether there is left-to-right or right-to-left traffic.

(6) As a general practice, the use of DO NOT ENTER and WRONG WAY signs is recommended at locations where the median width is 6 m (20 ft) and greater; consideration should also be given to the use of these signs for median widths narrower than 6 m (20 ft), where engineering judgment indicates a special need.

The rationale and supporting evidence for these recommendations can be found beginning on page 90 of this Handbook.

L. Design Element: Stop- and Yield-Controlled hatersection *Signage*

System-wide recommendations* to improve the safe use of intersections by older drivers, where the need for stop control or yield control has already been determined, include the following:

(1) The use of standard size (750 mm [30 in]) STOP (R1-1) and standard size (900 mm [36 in]) YIELD (R1-2) signs, as a minimum, is recommended wherever these devices are implemented.

(2) A minimum in-service sign background (red area) retroreflectivity level of 12 cd/m²/lux for roads with operating speeds under 64 km/h (40 mi/h), and 24 cd/m²/lux for roads with operating speeds of 64 km/h (40 mi/h) or higher, is recommended for STOP (R1-1) and YIELD (R1-2) signs.

*L. Design Element: Stop- and Yield-Controlled Intersection Signage**
(Continued)

(3) **The use of a supplemental warning sign panel mounted below the STOP (R1-1) sign, as illustrated, is recommended for two-way stop-controlled intersection sites selected on the basis of accident experience; where the sight triangle is restricted; and wherever a conversion from four-way stop to two-way stop operations is implemented.**

(4) **It is recommended that a STOP AHEAD sign (W3-1a) be used where the distance at which the STOP sign is visible is less than the AASHTO stopping sight distance (SSD) at the operating speed, plus an added preview distance of at least 2.5 s. Consideration should also be given to the use of transverse pavement striping or rumble strips upstream of stop-controlled intersections where engineering judgment indicates a special need due to sight restrictions, high approach speeds, or other geometric or operational characteristics likely to violate driver expectancy.**

***It is recognized that these broad recommendations may not address all of the diverse and varying problems occurring at any unique location, resulting in the need for engineering study to identify specific additional measures or combinations of measures to modify problem driver behaviors.**

The rationale and supporting evidence for these recommendations can be found beginning on page 96 of this Handbook.

M. Design Element: Devices for Lane Assignment on Intersection Approach

(1) The consistent placement of lane-use control signs (R3-5, R3-6) overhead on the signal mast arm at intersections is recommended, as a supplement to pavement markings and shoulder- and/or median-mounted signage.

(2) The consistent posting of lane-use control signs plus application of lane-use arrow pavement markings at a preview distance of at least 5 s (at operating speed) in advance of a signalized intersection is recommended, regardless of the specific lighting, channelization, or delineation treatments implemented at the intersection. Signs should be mounted overhead wherever practical, but they may be shoulder- and/or median-mounted in other cases.

The rationale and supporting evidence for these recommendations can be found beginning on page 103 of this Handbook.

N. Design Element: Traffic Signal Performance Issues

(1) To accommodate the increased optical density (reduced ocular transmittance) of the older driver's eye, and to improve availability of signal information under divided attention conditions during an intersection approach, it is recommended for all over-the-road signals that the Commission Internationale de l'Éclairage (CIE) 1980 standard for vertical intensity distribution (percent of peak) for a 300-mm (12-in) signal be adhered to in the United States, as given below:

Vertical Angle (degrees)	Intensity (% of Peak) Backplate	No Backplate
0–1.5	100	100
1.5–2.0	67	95
2.0–3.0	33	90
3.0–4.0	25	80
4.0–5.0	17	60
5.0–10.0	8	30

N. Design Element: Traffic Signal Performance Issues (Continued)

(2) To accommodate age differences in perception-reaction time (PRT), it is recommended that an all-red clearance interval be consistently implemented, with length determined according to the Institute of Transportation Engineers (1992) expressions given below.

When there is no pedestrian traffic, use:

$$r = \frac{W + L}{V}$$

Where there is the probability of pedestrian crossings, use the greater of:

$$r = \frac{P + L}{V} \quad \text{or} \quad r = \frac{P}{V}$$

Where there is significant pedestrian traffic or pedestrian signals to protect the crosswalk, use:

$$r = \frac{P + L}{V}$$

where:

r = length of red clearance interval, to the nearest 0.1 s.
W= width of intersection (ft or m), measured from the near-side stop line to the far edge of the conflicting traffic lane along the actual vehicle path.
P= width of intersection (ft or m), measured from the near-side stop line to the far side of the farthest conflicting pedestrian crosswalk along the actual vehicle path.
L= length of vehicle, recommended as 20 ft or 6.1 m.
V= speed of the vehicle through the intersection (ft/s).

N. Design Element: Traffic Signal Performance Issues (Continued)

(3) Wherever practical, the use of a backplate with traffic signals on roads with operating speeds of 64 km/h (40 mi/h) or less is recommended.

(4) The consistent use of a backplate with traffic signals on roads with operating speeds over 64 km/h (40 mi/h) is recommended.

The rationale and supporting evidence for these recommendations can be found beginning on page 107 of this Handbook.

0. Design Element: Fixed Lighting Installations

(1) Wherever feasible, fixed lighting installations are recommended (a) where the potential for wrong-way movements is indicated through accident experience or engineering judgment; (b) where pedestrian volumes are high; or (c) where shifting lane alignment, turn-only lane assignment, or a pavement-width transition forces a path-following adjustment at or near the intersection.

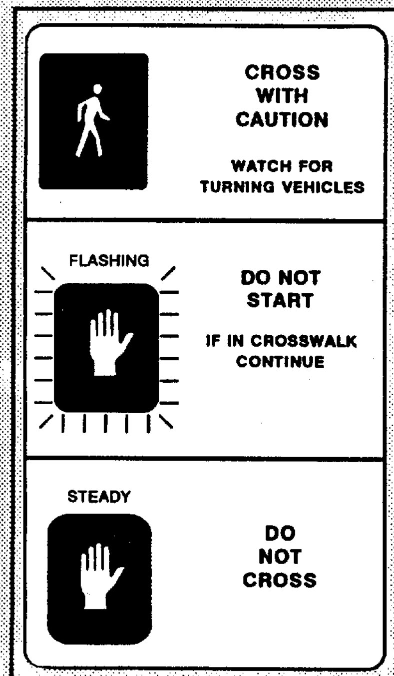
(2) Regular cleaning of lamp lenses and lamp replacement when output has degraded by 20 percent or more of peak performance (based on hours of service and manufacturer's specifications) are recommended for all fixed lighting installations at intersections.

The rationale and supporting evidence for these recommendations can be found beginning on page 115 of this Handbook.

P. Design Element: Pedestrian Control Devices

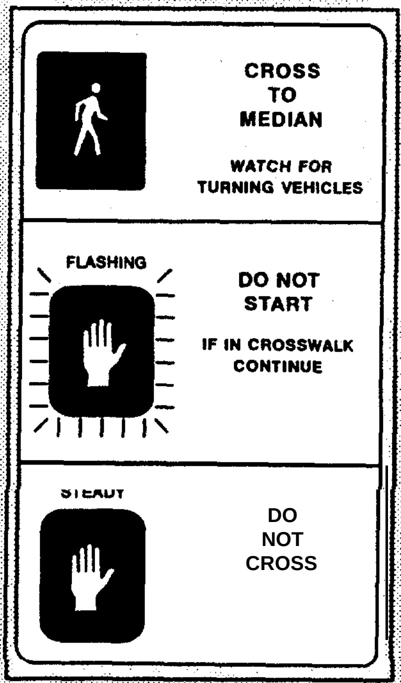
(1) To accommodate the shorter stride and slower gait of less capable (15th percentile) older pedestrians, and their exaggerated "start-up" time before leaving the curb, pedestrian control signal timing based on an assumed walking speed of 0.85 m/s (2.8 ft/s) is recommended.

(2) It is recommended that a placard explaining pedestrian control signal operations and presenting a warning to watch for turning vehicles be posted at the near corner of all intersections with a pedestrian crosswalk, using the design shown.



P. Design Element: Pedestrian Control Devices (Continued)

(3) It is recommended that at intersections where pedestrians cross in two stages using a median refuge island, the placard depicted in Recommendation P(2) be placed on the median refuge island, and that a placard modified as shown be placed on the near corner of the crosswalk.



The rationale and supporting evidence for these recommendations can be found beginning on page 118 of this Handbook.

II. INTERCHANGES (GRADE SEPARATION)

Background and *Scope* of *Handbook* Recommendations

Overall, freeways are characterized by the highest safety level (lowest fatality rates) when compared with other types of highways in rural and urban areas (American Automobile Association Foundation for Traffic Safety, 1995). At the same time, freeway interchanges have design features that have been shown to result in significant safety and operational problems. Taylor and McGee (1973) reported more than 20 years ago that erratic maneuvers are a common occurrence at freeway exit ramps and that the number of accidents in the vicinity of the exit ramp is four times greater than at any other freeway location. Two decades later, Lunenfeld (1993) reiterated that most freeway accidents and directional uncertainty occur in the vicinity of interchanges.

Distinct patterns in the occurrence of freeway interchange accidents emerge in studies that look specifically at driver age. Staplin and Lyles (1991) conducted a statewide (Michigan) analysis of the accident involvement ratios and types of violations for drivers in the following age groups: age 76 and older; ages 56-75, ages 27-55, and age 26 and younger. Using induced-exposure methods to gauge accident involvement levels, this analysis showed that drivers over age 75 were overrepresented as the driver at fault in merging and weaving accidents near interchange ramps. With respect to violation types, the older driver groups were cited most frequently for failing to yield and for improper use of lanes. Similarly, Harkey , Huang , and Zegeer's (1996) study of the precrash maneuvers and contributing factors in older driver freeway accidents indicated that older drivers' failure to yield was the most common contributing factor. These data raise concerns about the use of freeway interchanges by older drivers, in light of evidence presented by Lerner and Ratté (1991) that a dramatic growth in older driver freeway travel occurred between 1977 and 1988, with this trend expected to continue.

Age differences in interchange accidents and violations may be understood in terms of driving task demands and age-related diminished driver capabilities. The exit gore area is a transitional area that requires a major change in tracking. A driver (especially in an unfamiliar location) must process a large amount of directional information during a short period of time and at high speeds, while maintaining or modifying his/her position within the traffic stream. When drivers must perform guidance and navigation tasks in close proximity, the chances increase that a driver will become overloaded and commit errors (Lunenfeld, 1993). Erratic maneuvers resulting from driver indecisiveness in such situations include encroaching on the gore area, and even backing up on the ramp or the through lane. When weaving actions are required, the information-processing task demands for freeway interchange maneuvers-both entry and exit-are further magnified.

On a population basis, the age-related diminished capabilities that contribute most to older drivers' difficulties at freeway interchanges include losses in vision and information-processing ability, and decreased physical flexibility in the neck and upper body. Specifically, older adults show declines in static and dynamic acuity, increased sensitivity to glare, poor night vision, and reduced contrast sensitivity (McFarland, Domey, Warren, and Ward, 1960; Weymouth, 1960; Richards, 1972; Pitts, 1982; Sekuler, Kline, and Dismukes, 1982; Owsley, Sekuler, and Siemsen, 1983). These sensory losses are compounded by the following perceptual and cognitive deficits, the first two of which are recognized as being especially critical to safety: reduction in the ability to rapidly localize the most relevant stimuli in a driving scene, reduction in the ability to efficiently switch attention between multiple targets, reduction in working memory capacity, and reduction in processing speed (Avolio, Kroeck, and Panek, 1985; Plude and Hoyer, 1985; Ponds, Brouwer, and van Wolffelaar, 1988; Brouwer, Ickenroth, Ponds, and van Wolffelaar, 1990; Brouwer, Waterink, van Wolffelaar, and Rothengatter, 1991). The most important physical losses are reduced range of motion (head and neck), which impairs visual search, and slowed response time to execute a vehicle control movement, especially when a sequence of movements-such as braking, steering, accelerating to weave and then exit a freeway-is required (Smith and Sethi, 1975; Goggin, Stelmach, and Amrhein, 1989; Goggin and Stelmach, 1990; Hunter-Zaworski, 1990; Staplin, Lococo, and Sim, 1990; Ostrow, Shaffron, and McPherson, 1992).

One result of these age-related diminished capabilities is demonstrated by a driver who waits when merging and entering freeways at on-ramps until he/she is alongside traffic, then relies on mirror views of overtaking vehicles on the mainline to begin searching for an acceptable gap (McKnight and Stewart, 1990). Exclusive use of mirrors to check for gaps and slowing or stopping to look for a gap increase the likelihood of accidents and have a negative effect on traffic flow. Malfetti and Winter (1987), in a critical incident study of merging and yielding problems, reported that older drivers on freeway acceleration lanes merged so slowly that traffic was disrupted, or they stopped completely at the end of the ramp instead of attempting to approach the speed of the traffic flow before entering it. In Lerner and Ratté's (1991) research, older drivers in focus group discussions commented that they experienced difficulty maintaining vehicle headway because of slower reaction times, difficulty reading signs because of visual deficits, fatigue, mobility limitations, a tendency to panic or become disoriented, and loss of daring or confidence. Merging onto the freeway was the most difficult maneuver discussed during the focus group sessions. Needed improvements identified by these older drivers included the elimination of weaving sections and short merge areas, which would facilitate the negotiation of on-ramps at interchanges. Improvements identified to ease the exit process included better graphics, greater use of sign panels listing several upcoming exits, and other methods to improve advance signing for freeway exits.

This section will provide recommendations for highway design elements in four areas to enhance the performance of diminishedcapacity drivers at interchanges: A. exit signing and exit ramp gore delineation; B. acceleration/deceleration lane design features; C. fixed lighting installations; and D. traffic control devices for prohibited movements on freeway ramps.

The *Handbook* recommendations that follow are supported by material presented later in the "Rationale and Supporting Evidence" section under the "Interchanges (Grade Separation)" heading.

Recommendations by Design Element

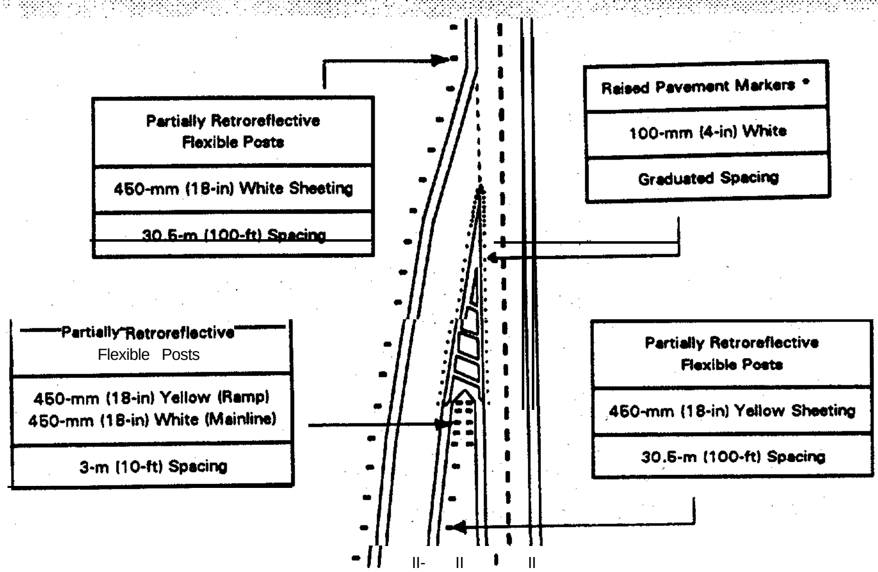
A. Design Element: Exit Signing and Exit Ramp Gore Delineation

(1) The calculation of letter size requirements for exit signing based on an assumption of *not more than* 10 m (33 ft) of legibility distance for each 25 mm (1 in) of letter height is recommended, for new or reconstructed installations and at the time of sign replacement.

(2) It is recommended that the MUTCD (Federal Highway Administration, 1988) requirements for multiple advance signing upstream of major and intermediate interchanges (section 2E-26) be extended to minor interchanges as well.

(3) A modification of diagrammatic guide signing displayed in the MUTCD (figure 2-30), such that the number of arrow shafts appearing on the sign matches the number of lanes on the roadway at the sign's location, is recommended for new or reconstructed installations.

(4) It is recommended that delineation in the vicinity of the exit gore at nonilluminated and partially illuminated interchanges include, *as a minimum*, the treatments illustrated in the figure below:



*** Snowplowable raised pavement markers may be used where appropriate for conditions.**

The rationale and supporting evidence for this recommendation can be found beginning on page 125 of this Handbook.

B. Design Element: Acceleration/Deceleration Lane Design Features

(1) It is recommended that acceleration lane lengths be determined using the higher of AASHTO (1994) table X-4 speed change lane criteria or NCHRP 3-35 (Reilly, Pfefer, Michaels, Polus, and Schoen, 1989) values for a given set of operational and geometric conditions, and assuming a 64 km/h (40 mi/h) ramp speed at the beginning of the gap search and acceptance process.

(2) A parallel versus a taper design for entrance ramp geometry is recommended.

(3) It is recommended that post-mounted delineators and/or chevrons be applied to delineate the controlling curvature on exit ramp deceleration lanes.

(4) It is recommended that AASHTO (1994) decision sight distance values be consistently applied in locating ramp exits downstream from sight-restricting vertical or horizontal curvature on the mainline (instead of locating ramps based on stopping sight distance [SSD] or modified SSD formulas).

The rationale and supporting evidence for this recommendation can be found beginning on page 133 of this Handbook.

C. Design Element: Fixed Lighting Installations

(1) Complete interchange lighting (CIL) is the preferred practice, but where a CIL system is not feasible to implement, a partial interchange lighting (PIL) system comprised of two high-mast installations per ramp—one fixture on the inner ramp curve near the gore and one fixture on the outer curve of the ramp, midway through the controlling curvature—is recommended.

The rationale and supporting evidence for this recommendation can be found beginning on page 142 of this Handbook.

D. *Design Element: Traffic Control Devices for Prohibited Movements on Freeway Ramps*

(1) The consistent use of 1,200 mm x 750 mm (48 in x 30 in) FREEWAY ENTRANCE signs for positive guidance as described as an option in section 2E-40 of the MUTCD (Federal Highway Administration, 1988), using a minimum letter height of 200 mm (8 in) with series D or wider font, is recommended.

(2) Where adjacent entrance and exit ramps intersect with a crossroad, the use of a median separator is recommended, with the nose of the separator delineated with yellow reflectorized paint and extending as close to the crossroad as practical without obstructing the turning path of vehicles. In addition, it is recommended that a KEEP RIGHT (R4-7a) sign be posted on the median separator nose.

(3) Where DO NOT ENTER (R5-1) and WRONG WAY (R5-9) signs are placed, in accordance with sections 2A-31 and 2E-40 of the MUTCD, a minimum size for R5-1 of 900 mm x 900 mm (36 in x 36 in) and for R5-9 of 1,200 mm x 800 mm (48 in x 32 in) is recommended, with corresponding increases in letter sizes, and the use of high-intensity sheeting. In addition, a mounting height (from the pavement to the bottom of the bottom sign) of between 450 mm and 900 mm (18 in and 36 in) is recommended, using the lowest value for this range that is practical when the presence of snow or other obstructions is taken into consideration.

(4) The application of 7.3-m (24-ft) wrong-way arrow pavement markings (see MUTCD section 2B-20) near the terminus on all exit ramps, accompanied by red raised-pavement markers facing wrong-way traffic, is recommended.

The rationale and supporting evidence for this recommendation can be found beginning on page 146 of this Handbook.

III. ROADWAY CURVATURE AND PASSING ZONES

Background and *Scope* of *Handbook* Recommendations

Accidents on horizontal curves have been recognized as a considerable safety problem for many years. Accident studies indicate that roadway curves experience a higher accident rate than tangents, with rates ranging from one-and-a-half to three to four times higher than tangents (Glennon, Neuman, and Leisch, 1985; Zegeer, Stewart, Reinfurt, Council, Neuman, Hamilton, Miller, and Hunter, 1990; Neuman, 1992). Lemer and Sedney (1988) reported anecdotal evidence that horizontal curves present problems for older drivers. Also, Lyles' (1993) analyses of accident data in Michigan found that older drivers are much more likely to be involved in accident situations where the drivers were driving too fast for the curve or, more significantly, were surprised by the curved alignment. In a review of the literature aimed at modifying driver behavior on rural road curves, Johnston (1982) reported that horizontal curves that are below 600 m (1,968 ft) in radius on two-lane rural roads, and those requiring a substantial reduction in speed from that prevailing on the preceding tangent section, were disproportionately represented among accident sites.

Successful curve negotiation depends upon the choice of appropriate approach speed and adequate lateral positioning through the curve. Many studies have shown that loss-of-control accidents result from an inability to maintain lateral position through the curve because of excessive speed, with inadequate deceleration in the approach zone. These problems in turn stem from a combination of factors, including poor anticipation of vehicle control requirements, induced by the driver's prior speed, and inadequate perception of the demands of the curve.

Many studies report a relationship between horizontal curvature (and the degree of curvature) and the total percentage of accidents by geometric design feature on the highways. The reasons for these accidents are related to the following inadequate driving behaviors:

- Deficient skills in negotiating curves, especially those of more than 3 degrees (Eckhardt and Flanagan, 1956).
- Exceeding the design speed on the curve (Messer, Mounce, and Brackett, 198 1).
- Exceeding the design of the vehicle path (Glennon and Weaver, 1971; Good, 1978).
- Failure to maintain appropriate lateral position in the curve (McDonald and Ellis, 1975).
- Incorrect anticipatory behavior of curve speed and alignment when approaching the curve (Messer et al., 1981; Johnston, 1982).
- Inadequate appreciation of the degree of hazard associated with a given curve (Johnston, 1982).

With respect to vertical curves, design policy is based on the need to provide drivers with adequate stopping sight distance (SSD). That is, enough sight distance must exist to permit drivers to see an obstacle soon enough to stop for it under some set of reasonable worst-case conditions, The parameters that determine sight distance on crest vertical curves include the change of grade, the length of the curve, the height above the ground of the driver's eye, and the height of the obstacle to be seen. SSD is determined by reaction time, speed of vehicle, and tire-pavement coefficient of friction. There is some concern with the validity of the SSD model that has been in use for over 50 years, however. Current practice assumes an obstacle height of 150 mm (6 in) and a locked-wheel, wet-pavement stop. Minimum lengths of crest vertical curves are based on sight distance and driver comfort. These criteria do ***not*** currently include adjustments for age-related effects in driving performance measures, which would suggest an even more conservative approach. At the same time, the general lack of empirical data demonstrating benefits for limited sight distance countermeasures has led some to propose liberalization of model criteria, such as obstacle height.

Standards and criteria for sight distance, horizontal and vertical alignment, and associated traffic control devices are based on the following driver performance characteristics: detection and recognition time, perception-reaction time, decision and response time, time to perform brake and accelerator movements, maneuver time, and (if applicable) time to shift gears. However, these values have typically been based on driving performance (or surrogate driving measures) of the entire driving population, or have been formulated from research biased toward younger (college-age) as opposed to older driver groups. The models underlying these design standards and criteria therefore have not, as a rule, included variations to account for slower reaction time or other performance deficits consistently demonstrated in research on older driver response capabilities. In particular, diminished visual performance (reduced acuity and contrast sensitivity), physical capability (reduced strength to perform control movements and sensitivity to lateral force), cognitive performance (attentional deficits and declines in choice reaction time in responses to unpredictable stimuli), and perceptual abilities (reduced accuracy of processing speed-distance information as required for gap judgments) combine to make the task of negotiating the highway design elements addressed in this section more difficult and less forgiving for older drivers.

This ‘section will provide recommendations to enhance the performance of diminished capacity drivers as they negotiate roadway curvature and passing zones, focusing on four design elements: A. pavement markings and delineation on horizontal curves; B. pavement width on horizontal curves; C. crest vertical curve length and advance signing for sight-restricted locations; and D. passing zone length, passing sight distance, and passing/overtaking lanes on two-lane highways.

The ***Handbook*** recommendations that follow are supported by material presented later in the “Rationale and Supporting Evidence” section under the “Roadway Curvature and Passing Zones” heading.

Recommendations by Design Element

A. Design Element: Pavement Markings and Delineation on Horizontal Curves

(1) The installation and maintenance of white edgelines of normal width (MUTCD [Federal Highway Administration, 1988]) on horizontal curves at an effective luminance (L) contrast level of at least 5.0 is recommended on all highways (including arterials) without median separation of opposing directions of traffic, where

$$\textit{luminance contrast} = \frac{L_{stripe} - L_{pavement}}{L_{pavement}}$$

(2) A minimum in-service contrast value of 3.75 is recommended for pavement edgelines on horizontal curves where median barriers effectively block drivers' view of oncoming headlights or where median width exceeds 15 m (49 ft).

(3) The installation (at standard spacing) of centerline raised-pavement markers beginning 5 s driving time (at 85th percentile speed) before, and continuing through the length of, all horizontal curves of radii under 1,000 m (3,281 ft) is recommended.

(4) The installation of roadside delineation devices at a maximum spacing (S) of 12 m (40 ft) on all horizontal curves with a radius (R) of 185 m (600 ft) or less, is recommended; for curves of radii over 185 m (600 ft), the standard/MUTCD formula (in feet) to define roadside delineator spacing intervals is recommended, where

$$S = 3\sqrt{R - 50}$$

The rationale and supporting evidence for this recommendation can be found beginning on page 151 of this Handbook.

B. Design Element: Pavement Width on Horizontal Curves

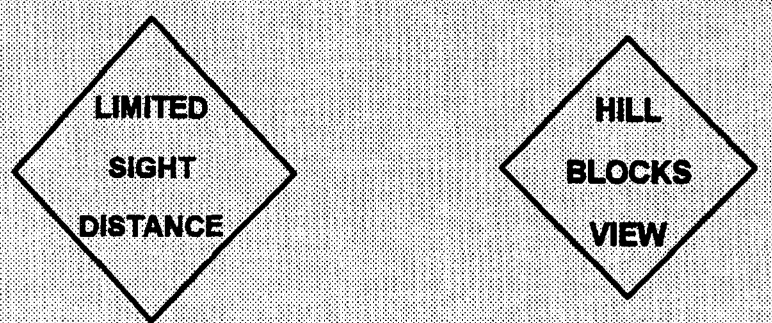
(1) A minimum lane-plus-paved-shoulder width of 5.5 m (18 ft) through the length of arterial horizontal curves ≥ 3 degrees of curvature is recommended (assuming AASHTO [1994] design values for superelevation and coefficient of side friction).

The rationale and supporting evidence for this recommendation can be found beginning on page 157 of this Handbook.

C. Design Element: Crest Vertical Curve Length and Advance Signing for Sight-Restricted Locations

(1) To accommodate the exaggerated decline among older drivers in response to unexpected hazards, it is recommended that the present criterion of 150 mm (6 in) for obstacle height on crest vertical curves be preserved in the design of new and reconstructed facilities.

(2) Where a need has already been determined for installation or replacement of a LIMITED SIGHT DISTANCE (W14-4) sign, the alternate message HILL BLOCKS VIEW is recommended, using the special sign size of 900 mm x 900 mm (36 in x 36 in) cited in *Standard Highway Signs as Specified in the Manual on Uniform Traffic Control Devices* (Federal Highway Administration, 1979) as a minimum.



not recommended ***recommended***

C. Design Element: *Crest Vertical Curve Length and Advance Signing for Sight-Restricted Locations* (Continued)

(3) If a signalized intersection is obscured by vertical or horizontal curvature in a manner that the signal condition becomes visible at a preview distance of 8 s or less (at operating speed), then the use of the advance warning sign PREPARE TO STOP, with a yellow flasher activated during the red-signal phase, is recommended.

The rationale and supporting evidence for this recommendation can be found beginning on page 160 of this Handbook.

D. Design Element: *Passing Zone Length, Passing Sight Distance, and Passing/Overtaking Lanes on Two-Lane Highways*

(1) A minimum passing zone length of 350 m (1,150 ft) is recommended for any facility with an operating speed of 64 km/h (40 mi/h) or greater.

(2) A minimum passing sight distance (MUTCD definition [Federal Highway Administration, 1988]) of 215 m (705 ft) is recommended for any facility with an operating speed of 64 km/h (40 mi/h) or greater.

(3) Use of special size (1,200 mm x 1,600 mm x 1,600 mm [48 in x 64 in x 64 in]) NO PASSING ZONE pennant (W14-3) as a high-conspicuity supplement to conventional centerline pavement markings at the beginning of no passing zones is recommended.

(4) To the extent feasible for new or reconstructed facilities, excepting those with low traffic volume, the implementation of passing/overtaking lanes (in each direction) at intervals of no more than 5 km (3.1 mi) is recommended.

The rationale and supporting evidence for this recommendation can be found beginning on page 164 of this Handbook.

IV. CONSTRUCTION/WORK ZONES

Background and Scope of *Handbook* Recommendations

Highway construction and maintenance zones deserve special consideration with respect to older driver needs because of their strong potential to violate driver expectancy. Alexander and Lunenfeld (1986) properly emphasized that driver expectancy is a key factor affecting the safety and efficiency of all aspects of the driving task. Consequently, it is understandable that accident analyses consistently show that more accidents occur on highway segments containing construction zones than on the same highway segments before the zones were implemented (Juergens, 1972; Graham, Paulsen, and Glennon, 1977; Lisle, 1978; Nemeth and Migletz, 1978; Paulsen, Harwood, and Glennon, 1978; Garber and Woo, 1990; Hawkins, Kacir, and Ogden, 1992).

Work zone traffic control must provide adequate notice to motorists describing the condition ahead, the location, and the required driver response. Once drivers reach a work zone, pavement markings, signing, and channelization must be conspicuous and unambiguous in providing guidance through the area. The National Transportation Safety Board (NTSB) believes that the MUTCD guidelines concerning signing and other work zone safety features provide more than adequate warning for a *vigilant* driver, but may be inadequate for an inattentive or otherwise impaired driver (NTSB, 1992). It is within this context that functional deficits associated with normal aging, as described below, may place older drivers at greater risk when negotiating work zones.

In an accident analysis at 20 case-study work zone locations, among the most frequently listed contributing factors were driver attention errors and failure to yield the right-of-way (Pigman and Agent, 1990). Older drivers are most likely to demonstrate these deficits. Research on selective attention has documented that older adults respond much more slowly to stimuli that are unexpected (Hoyer and Familant, 1987), suggesting that older adults could be particularly disadvantaged by changes in roadway geometry and operations characteristic of construction zones. There is also research indicating that older adults are more likely to respond to new traffic patterns in an "automatized" fashion, resulting in more frequent driver errors (Fisk, McGee, and Giambra, 1988). To respond in situations that require decisions among multiple and/or unfamiliar alternatives, with unexpected path-following cues, drivers' actions are described by *complex reaction times* that are longer than reaction times in simple situations with expected cues. In Mihal and Barrett's (1976) analysis relating simple, choice, and complex reaction time to crash involvement, only an increase in complex reaction time was associated with accidents. The relationship with driver age was most striking: the correlation between complex reaction time and accident involvement increased from 0.27 for the total analysis sample (all ages) to 0.52 when only older adults were included. Such data suggest that in situations where there is increased complexity in the information to be processed by drivers—such as work zones—the most relevant information must be communicated in a dramatic manner to ensure that it receives a high priority by older individuals.

Compounding their exaggerated difficulties in allocating attention to the most relevant aspects of novel driving situations, diminished visual capabilities among older drivers are well documented (McFarland, Domey, Warren, and Ward, 1960; Weymouth, 1960; Richards, 1972;

Pitts, 1982; Sekuler, Kline, and Dismukes, 1982; Owsley, Sekuler, and Siemsen, 1983; Wood and Troutbeck, 1994). Deficits in static and dynamic acuity and contrast sensitivity, particularly under low luminance conditions, make it more difficult for them to detect and read traffic signs, to read variable message signs, and to detect pavement markings and downstream channelization devices. Olson (1988) determined that for a traffic sign to be noticed at night in a visually complex environment, its reflectivity must be increased by a factor of 10 to achieve the same level of conspicuity as in a low-complexity environment. Mace (1988) asserted that the minimum required visibility distance—the distance from a traffic sign required by drivers in order to detect, understand, make a decision, and complete a vehicle maneuver before reaching a sign—is increased significantly for older drivers due to their poorer visual acuity and contrast sensitivity, coupled with inadequate sign luminance and legend size. Other age-related deficits cited by Mace (1988) include lowered driver alertness, slower detection time in complex roadway scenes due to distraction from irrelevant stimuli, increased time to understand unclear messages such as symbols, and slower decisionmaking.

This section will provide recommendations to enhance the performance of diminished-capacity drivers as they approach and travel through construction/work zones, keyed to five specific design elements: A. advance signing for lane closure(s); B. variable (changeable) message signing practices; C. channelization practices; D. delineation of crossovers/alternate travel paths; and E. temporary pavement markings.

The *Handbook* recommendations that follow are supported by material presented later in the "Rationale and Supporting Evidence" section under the "Construction/Work Zones" heading.

Recommendations by Design Element

A. Design Element: Advance Signing for Lane Closure(s)

(1) At construction/maintenance work zones on high-speed and divided highways, the consistent use of a flashing arrow panel located at the taper for each lane closure is recommended.

(2) In implementing advance signing for lane closures as per MUTCD Part VI (Federal Highway Administration, 1988), it is recommended that a supplemental (portable) variable message sign displaying the one-page (phase) message LEFT (RIGHT, CENTER) LANE CLOSED be placed 800–1,600 m (2,625–5,250 ft) upstream of the lane closure taper.

or

Redundant static signing should be used, where both the first upstream sign (e.g., W20-1) and the second sign (e.g., W20-5) encountered by the driver are equipped with flashing warning lights throughout the entire period of the lane closure, and a minimum letter height of 200 mm (8 in) is used.

The rationale and supporting evidence for this recommendation can be found beginning on page 169 of this Handbook.

B. Design Element: Variable (Changeable) Message Signing Practices

(1) It is recommended that no more than two phases be used on a variable message sign (VMS); if a message cannot be conveyed in two phases, multiple VMS's and/or a supplemental highway advisory radio message should be used.

(2) It is recommended that no more than one unit of information (defined in "Rationale and Supporting Evidence" text for this section) should be displayed on a single line on a VMS, and no more than three units should be displayed on any single phase.

(3) For nondiversion VMS messages split into two phases, a total of no more than four *unique* units of information should be presented.

B. Design Element: Variable (Changeable) Message Signing Practices (Continued)

(4) Generally, to display information about accidents, road construction, or environmental warnings on permanent variable message signs, line 1 should present the problem, line 2 the location or distance ahead, and line 3 the recommended driver action. This is the standard for single-phase messages: *problem | location (or distance ahead) | action advised/required.*

(5) When a portable variable message sign is used to display a message in two phases, the problem and location statements should be displayed during phase 1 and the effect or action statement during phase 2. For example, phase 1 could read:

ROADWORK | NEXT | 2 MILES

while phase 2 could read:

LEFT | LANE | CLOSED

If legibility distance restrictions rule out a two-phase display, the use of abbreviations plus elimination of the problem statement is the recommended strategy to allow for the presentation of the entire message on one phase:

LFT LANE | CLOSED | NEXT 2 MI

(6) Where abbreviations are necessary in VMS operations, an adherence to the "acceptable," "not acceptable," and "acceptable with prompt" categories published by Dudek, Huchingson, Williams, and Koppa (1981) and reproduced in the "Rationale and Supporting Evidence" text for this section is recommended.

The rationale and supporting evidence for this recommendation can be found beginning on page 173 of this Handbook.

C. Design Element: Channelization Practices

(1) The following minimum dimensions for channelizing devices used in highway work zones are recommended, to accommodate the needs of older drivers:

(1a) Traffic cones—900 mm (36 in) height (with at least a 300-mm [12-in] reflective collar for nighttime operations).

(1b) Traffic tubes—1,050 mm (42 in) height (with at least a 300-mm [12-in] reflective band for nighttime operations).

(1c) Vertical panels—300 mm (12 in) width.

(1d) Barricades—300 mm x 900 mm (12 in x 36 in) minimum dimension.

(2) The use of a flashing arrow panel at the start of the taper at all right and left lane closures is recommended on all roadways with an operating speed of 72 km/h (45 mi/h) and greater. On lower speed roadways *without an arrow panel*, it is recommended that the start of the taper for a lane closure be marked with a reflectorized plastic drum with steady-burn light, and accompanying chevrons, as a channelizing treatment.

(3) The spacing of channelizing devices (in feet) through a work zone and through taper and transition sections at not more than the speed limit (in miles per hour) is recommended, with spacing (in feet) through the taper for a lane closure at not more than one-half the speed limit (in miles per hour) where engineering judgment indicates a special need for speed reduction.

(4) The use of side reflectors with cube-corner lenses on Jersey barriers and related concrete channelizing devices spaced (in feet) at not more than the construction zone speed limit (in miles per hour) through a work zone is recommended.

The rationale and supporting evidence for this recommendation can be found beginning on page 183 of this Handbook.

D. Design Element: Delineation of Crossovers/Alternate Travel Paths

(1) The use of positive barriers in transition zones, and positive separation (channelization) between opposing two-lane traffic throughout a crossover, is recommended for all roadway classes except residential.

(2) A minimum spacing (in feet) of one-half the construction zone speed limit (in miles per hour) for channelizing devices (other than concrete barriers) is recommended in transition areas, and through the length of the crossover and in the termination area downstream (where operations as existed prior to the crossover resume).

(3) The use of side reflectors with cube-corner lenses spaced (in feet) at not more than the construction zone speed limit (in miles per hour) on concrete channelizing barriers in crossovers (or alternately the use of retroreflective sheeting on plastic glare-control louvers [paddles] placed in crossovers) is recommended.

(4) It is recommended for construction/work zones on high-volume roadways that plastic glare-control louvers (paddles) be mounted on top of concrete channelizing barriers, when used in transition and crossover areas, at a spacing of not more than 600 mm (24 in).

The rationale and supporting evidence for this recommendation can be found beginning on page 188 of this Handbook.

E. Design Element: Temporary Pavement Markings

(1) Where temporary pavement markings shorter than the 3 m (10 ft) standard length are implemented, it is recommended that a raised pavement marker be placed at the center of the gap between successive markings.

or, if deemed a more cost-effective alternative:

It is recommended that temporary pavement markings shorter than 3 m (10 ft) be supplemented with devices including cones, tubes, or barrels placed on the centerline between opposing lanes, spaced (in feet) and maintained at not more than the construction zone speed limit (in miles per hour) apart.

The rationale and supporting evidence for this recommendation can be found beginning on page 193 of this Handbook.

RATIONALE AND SUPPORTING EVIDENCE

This section of the *Handbook* is organized in terms of the same classes of highway features as the Recommendations: I. Intersections (At-Grade), II. Interchanges (Grade Separation), III. Roadway Curvature and Passing Zones, and IV. Construction/Work Zones. Within each of these four classes, subsections are organized in terms of design elements with unique geometric, operational, and/or traffic control characteristics, also consistent with the Recommendations.

At the beginning of each subsection within a class of highway features, reference material for a particular design element is introduced using a cross-reference table. This table relates the discussion in that subsection—as well as the associated Recommendations, presented earlier—to entries in standard reference manuals consulted by practitioners in this area. Principal among these reference manuals are the *Manual on Uniform Traffic Control Devices* (Federal Highway Administration [FHWA], 1988); *Manual on Uniform Traffic Control Devices for Streets and Highways* [MUTCD], *Part VI: Standards and Guides for Traffic Controls for Street and Highway Construction, Maintenance, Utility, and Incident Management Operations* (FHWA, 1993); and the *Policy on Geometric Design of Highways and Streets* [the Green Book] (American Association of State Highway and Transportation Officials [AASHTO], 1994). Other standard references with more restricted applicability, which also appear in the cross-reference tables for selected design elements, include National Cooperative Highway Research Program (NCHRP) Report No. 279, *Intersection Channelization Design Guide* (Neuman, 1985), and the *Roadway Lighting Handbook, Chapter 6 Addendum* (FHWA, 1983).

Material in this part of the *Handbook* represents, to as great an extent as possible at the time of its development, the results of empirical work documenting older driver performance for the highway features of interest. Observational and controlled field studies were given precedence, together with laboratory simulations employing traffic stimuli and relevant situational cues. Accident data are cited as appropriate. In addition, though of lower priority, some citations reference pertinent findings from "basic" research on age differences in response capability that are tied logically to performance in highway settings.

I. INTERSECTIONS (AT-GRADE)

The following discussion presents the rationale and supporting evidence for *Handbook* recommendations pertaining to these 16 design elements (A–P):

A. Intersecting Angle (Skew)
B. Receiving Lane (Throat) Width for Turning Operations
C. Channelization
D. Intersection Sight Distance (Sight Triangle)
E. Opposite (Single) Left-Turn Lane Geometry, Signing, and Delineation
F. Edge Treatments/Delineation of Curbs, Medians, and Obstacles
G. Curb Radius
H. Traffic Control for Left-Turn Movements at Signalized Intersections
I. Traffic Control for Right-Turn/RTOR Movements at Signalized Intersections
J. Street-Name Signage
K. One-Way/Wrong-Way Signage
L. Stop- and Yield-Controlled Intersection Signage
M. Devices for Lane Assignment on Intersection Approach
N. Traffic Signal Performance Issues
O. Fixed Lighting Installations
P. Pedestrian Control Devices

A. Design Element: Intersecting Angle (Skew)

Table 1. Cross-references of related entries for intersecting angle (skew).

Applications in Standard Reference Manuals		
MUTCD (1988)	AASHTO Green Book (1994)	NCHRP 279 Intersection Channelization Design Guide (1985)
Pg. 3B-11, Para. 3	Pg. 426, Para. 5 Pg. 628, Item C.4 Pg. 630, Para. 1 Pgs. 643-645, Sect. on *Alignment* Pgs. 648-651, Tables IX-1 and IX-2 Pgs. 663-664, Sect. on *Oblique-Angle Turns* Pgs. 689-690, Sect. on *Oblique-Angle Turns with Corner Islands* Pg. 691, Table IX-4 Pgs. 718-720, Sect. on *Effect of Skew* Pgs. 764-767, Sect. on *Effect of Skew*	Pg. 19, Top Fig. Pg. 25, Para. 2 Pg. 30, Para. 1 and Top & Middle Fig(s). Pg. 31, Para. 3 & Bottom Left Fig. Pgs. 42-44, Sect. on *Angle of Intersection* Pg. 71, Top two Fig(s).

There is broad agreement that right-angle intersections are the preferred design. Decreasing the angle of the intersection makes detection of and judgments about potential conflicting vehicles on crossing roadways much more difficult. In addition, the amount of time required to maneuver through the intersection increases, for both vehicles and pedestrians, due to the increased pavement area. However, there is some inconsistency among reference sources concerning the degree of skew that can be safely designed into an intersection. The Green Book states that an angle of 60 degrees provides most of the benefits that are obtained with a right-angle intersection. Subsequently, factors to adjust intersection sight distances for skewness are suggested for use only when angles are less than 60 degrees (AASHTO, 1990). Another source on subdivision street design states that: "Skewed intersections should be avoided, and in no case should the angle be less than 75 degrees" (Institute of Transportation Engineers [ITE], 1984).

Skewed intersections pose particular problems for older drivers. Many older drivers experience a decline in head and neck mobility, which accompanies advancing age and may contribute to the slowing of psychomotor responses. Joint flexibility, an essential component of driving skill, has been estimated to decline by approximately 25 percent in older adults due to arthritis, calcification of cartilage, and joint deterioration (Smith and Sethi, 1975). A restricted range of motion reduces an older driver's ability to effectively scan to the rear and sides of his or her vehicle to observe blind spots, and similarly may be expected to hinder the timely recognition of conflicts during turning and merging maneuvers at intersections (Ostrow, Shaffron, and McPherson, 1992). For older drivers, diminished physical capabilities may affect their performance at intersections designed with acute angles by requiring them to turn their heads further than would be required at a right-angle intersection. This obviously creates more of a problem in determining appropriate gaps. For older pedestrians, the longer exposure time within the intersection becomes a major concern.

In a survey of older drivers conducted by Yee (1985), 35 percent of the respondents reported problems with arthritis and 21 percent indicated difficulty in turning their heads to scan rearward while driving. Excluding vision/visibility problems associated with nighttime operations, difficulty with head turning placed first among *all* concerns mentioned by older drivers participating in a more recent focus group conducted to examine problems in the use of intersections where the approach leg meets the main road at a skewed angle, and/or where channelized right-turn lanes require an exaggerated degree of head/neck rotation to check for traffic conflicts before merging (Staplin, Harkey, Lococo, and Tarawneh, 1997). Comments about this geometry centered around the difficulty older drivers experience turning their heads at angles less than 90 degrees to view traffic on the intersecting roadway, and several participants reported an increasing reliance on outside rearview mirrors when negotiating highly skewed angles. However, they reported that the outside mirror is of no help when the roads meet at the middle angles (e.g., 40 to 55 degrees) and a driver is not flexible enough to physically turn to look for traffic. In an observational field study conducted as a part of the same project, Staplin et al. (1997) found that approximately 30 percent of young/middle-aged drivers (ages 25–45) and young-old drivers (ages 65–74) used their mirrors in addition to making head checks before performing a right-turn-on-red (RTOR) maneuver at a skewed intersection (a channelized right-turn lane at a 65-degree skew). By comparison, none of the drivers age 75 and older used their mirrors; instead, they relied solely on information obtained from head/neck checks. In this same study, it was found that the likelihood of a driver making an RTOR maneuver is reduced by intersection skew angles that make it more difficult for the driver to view conflicting traffic.

The practical consequences of restricted head and neck movement on driving performance at T-intersections were investigated by Hunter-Zaworski (1990), using a simulator to present videorecorded scenes of intersections with various levels of traffic volume and sight distance in a 180-degree field of view from the driver's perspective. Drivers in two subject groups, ages 30–50 and 60–80, depressed a brake pedal to watch a video presentation (on three screens), then released the pedal when it was judged safe to make a left turn; half of *each* age group had a restricted range of neck movement as determined by goniometric measures of maximum (static) head-turn angle. Aside from demonstrating that skewed intersections are hazardous for any driver with an impairment in neck movement, this study found that maneuver decision time increased with both age *and* level of impairment. Thus, the younger drivers in this study were able to compensate for their impairments, but older drivers both with and without impairments were unable to make compensations in their (simulated) intersection response selections.

These research findings reinforce the desirability of providing a 90-degree intersection geometry and endorse the ITE (1984) recommendation establishing a 75-degree minimum as a practice to accommodate age-related performance deficits.

B. Design Element: Receiving Lane (Throat) Width for Turning Operations

Table 2. Cross-references of related entries for receiving lane (throat) width for turning operations.

Applications in Standard Reference Manuals	
AASHTO Green Book (1994)	**NCHRP 279 Intersection Channelization Design Guide (1985)**
Pg. 200, Para. 2 Pg. 647, Para. 2 Pg. 673, Para. 5 Pg. 676, Para(s) 3-5 Pgs. 749-751, Sect. on *Speed-Change Lanes at Intersections*	Pg. 57, Para. 5, 1st Bullet Pg. 58, Fig. 4-20

Design recommendations for lane width at intersections follow from consideration of vehicle maneuver requirements and their demands on drivers. Positioning a vehicle within the lane in preparation for turning has been rated as a critical task (McKnight and Adams, 1970). Swinging too wide to lengthen the turning radius and minimize rotation of the steering wheel ("buttonhook turn") is a common practice of drivers lacking strength (including older drivers) and physically limited drivers (McKnight and Stewart, 1990).

Two factors can compromise the ability of older drivers to remain within the boundaries of their assigned lanes during a left turn. One factor is the diminishing ability to share attention (i.e., to assimilate and concurrently process multiple sources of information from the driving environment). The other factor involves the ability to turn the steering wheel sharply enough, given the speed at which they are traveling, to remain within the boundaries of their lanes. Some older drivers seek to increase their turning radii by initiating the turn early and rounding-off the turn. The result is either to cut across the apex of the turn, conflicting with vehicles approaching from the left, or to intrude upon a far lane in completing the turn.

The Intersection Channelization Design Guide (Neuman, 1985) states that "left-turn lane widths should reflect the speed, volume, and vehicle mix. Although 3.6-m (12-ft) widths are desirable, lesser widths may function effectively and safely. Absolute minimum widths of 2.7 m (9 ft) should be used only in unusual circumstances, and only on low-speed streets with minor truck volumes." Similarly, the ITE (1984) guidelines suggest a minimum lane width of 3.3 m (11 ft) and specify 3.6 m (12 ft) as desirable. These guidelines suggest that wider lanes be avoided due to the resulting increase in pedestrian crossing distances. However, the ITE guidelines provide a range of lane widths at intersections from 2.7 m to 4.3 m (9 ft to 14 ft), where the wider lanes would be used to accommodate larger turning vehicles, which have turning paths that sweep a path from 4.1 m (13.6 ft) for a single-unit truck or bus, up to 6.3 m (20.6 ft) for a semitrailer.

Results of field observation studies conducted by Firestine, Hughes, and Natelson (1989) found that trucks performing turns on urban roads encroached into other lanes on streets with widths of less than 3.6 m (12 ft). They noted that on rural roads, lanes wider than 3.6 m or 4.0 m (12 ft or 13 ft) allowed oncoming vehicles to move further right to avoid trucks, and shoulders wider than 1.2 m (4 ft) allowed oncoming vehicles a greater margin of safety.

In an observational field study conducted to determine how older drivers (age 65 and older) compare with younger drivers during left-turn operations under varying intersection geometries, one variable that showed significant differences in older and younger driver behavior was turning path (Staplin, Harkey, Lococo, and Tarawneh, 1997). Older drivers encroached into the opposing lane of the cross street (see figure 1, turning path trajectory number 1) when making the left turn more often than younger drivers at the location where the throat width (equivalent to the lane width) measured 3.6 m (12 ft). At the location where the throat width measured 7 m (23 ft), which consisted of a 3.6-m (12-ft) lane and a 3.3-m (11-ft) shoulder, there was no significant difference in the turning paths. The narrower throat width resulted in higher encroachments by older drivers, who physically may have a harder time maneuvering their vehicles through smaller areas.

These data sources indicate that a 3.6-m (12-ft) lane width provides the most reasonable tradeoff between the need to accommodate older drivers, as well as larger turning vehicles, without penalizing the older pedestrian in terms of exaggerated crossing distance.

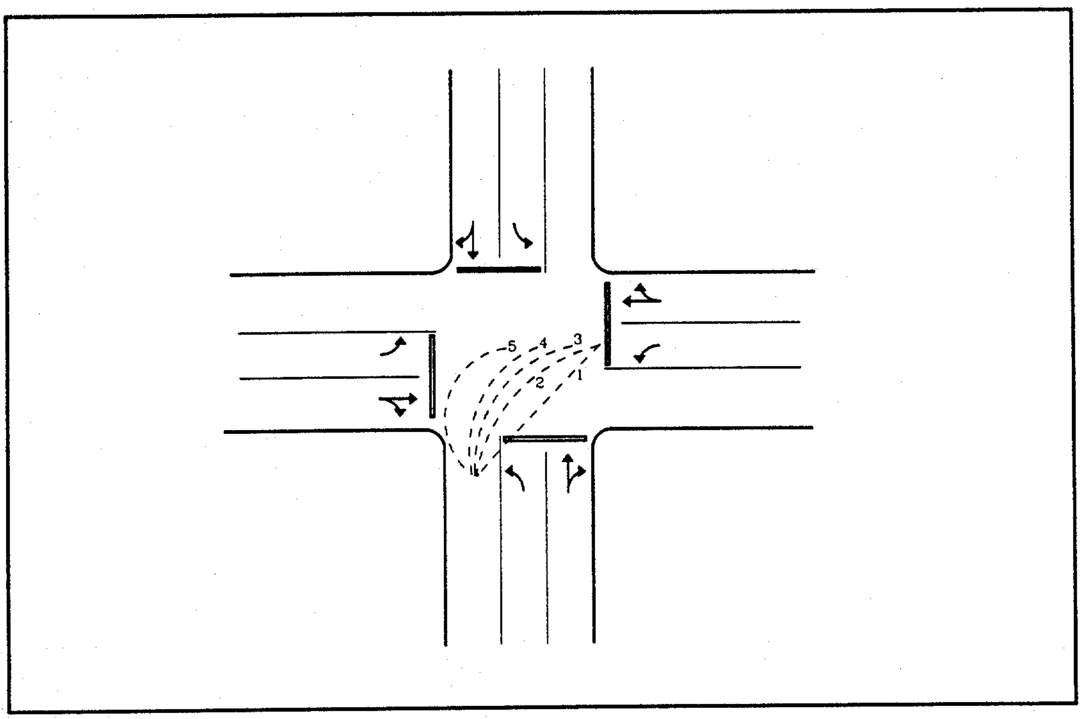


Figure 1. Turning path taken by left-turning vehicles, where 1 =encroach into opposing cross-traffic stream; 2, 3, and 4 = proper turning from different points within the intersection; and 5 = left turn from a position requiring a greater-than-90-degree turn to enter cross street.

C. Design Element: Channelization

Table 3. Cross-references of related entries for channelization.

Applications in Standard Reference Manuals			
MUTCD (1988)	**AASHTO Green Book (1994)**	**Roadway Lighting Handbook Chapter 6 (1983)**	**NCHRP 279 Intersection Channelization Design Guide (1985)**
Pg. 3B-14, Sect(s). on *Channelizing Line* and *Marking of Interchange Ramps* Pgs. 3b-15 - 3B-19, Figures 3-11 through 3-13 Pg. 3F-1, Sect. on *Channelizing Devices* Pg. 5A-2, Sect. on *Traffic Channelizing Islands*	Pgs. 631-632, Sect. on *Channelized Three-Leg Intersections* Pgs. 635-641, Sect. on *Channelized Four-Leg Intersections* Pgs. 674-678, Sect(s). on *Channelized Islands* and *Divisional Islands* Pgs. 680-687, Sect. on *Delineation and Approach-End Treatment* Pgs. 746-747, Para. 2 and Fig(s). IX-56 and IX-57. Pgs. 748-749, Sect. on *Channelization*	Pg. 18, Form 2 Pg. 21, Table 1 Pg. 26, Para. 2	Pg. 1, Para. 2-3 Pg. 18, Middle Fig. Pg. 21, Fig. 3-1 Pg. 24, Bottom Fig. Pg. 25, Para. 3 and Bottom Right Fig. Pg. 26, Top Fig. Pg. 27, Para(s) 2-3 Pg. 32, Middle Fig. Pg. 34, Para. 1 & Bottom Fig. Pg. 35, Bottom Left Fig. Pg. 74, Fig. 4-30 Pgs. 75-77, Sect(s). on *Guidelines for Selection of Island Type, Guidelines for Design of Traffic Islands,* and *Guidelines for Design of Median Islands* Pgs. 79-80, Fig(s) 4-34 & 4-35

The spatial visual functions of acuity and contrast sensitivity are important in the ability to detect/recognize downstream geometric features such as pavement width transitions, channelized turning lanes, island and median features across the intersection, and any nonreflectorized raised elements at intersections. Visual acuity (the ability to see high-contrast, high-spatial-frequency stimuli, such as black letters on a white eye chart) shows a slow decline beginning at approximately age 40, and marked acceleration at age 60 (Richards, 1972). Approximately 10 percent of men and women between ages 65 and 75 have acuity worse than 20/30, compared with roughly 30 percent over the age of 75 (Kahn, Leibowitz, Ganley, Kini, Colton, Nickerson, and Dawber, 1977). A driver's response to intersection geometric features is influenced in part by the processing of high-spatial-frequency cues—for example, the characters on upstream advisory signs—but it is the larger, often diffuse edges defining lane and pavement boundaries, curb lines, and raised median barriers that are the targets with the highest priority of detection for safety. Older persons' sensitivity to visual contrast (the ability to see objects of various shapes and sizes under varying levels of contrast) also declines beginning around age 40, then declines steadily as age increases (Owsley, Sekuler, and Siemsen, 1983). Poor contrast sensitivity has been shown to relate to increased crash involvement for drivers age 66 and older, when incorporated into a battery of vision tests also including visual acuity and horizontal visual field size (Decina and Staplin, 1993).

The effectiveness of channelization from a safety perspective has been documented in several studies. An evaluation of Federal Highway Safety Program projects showed channelization to produce an average benefit/cost ratio of 2.31 (Strate, 1980). One of the advantages of using curbed medians and intersection channelization is that it gives a better indication to motorists of the proper use of travel lanes at intersections. In a set of studies performed by the California Department of Public Works investigating the differences in accident experience with raised versus painted channelization, the findings were as follows: raised traffic islands are more effective than painted islands in reducing frequencies of night accidents, particularly in urban areas; and little difference is noted in the effectiveness of raised versus painted channelizing islands at rural intersections (Neuman, 1985).

One of the most common uses of channelization is for the separation of left-turning vehicles from the through-traffic stream. The reasons for designing intersections with left-turn lanes include: (1) proven safety effectiveness, (2) effectiveness in improving intersection capacity, (3) flexibility in possible signal phasing schemes, and (4) better understanding of intended traffic operations by the driving public. Guidance on when to include left-turn lanes varies with each State, as revealed in a survey of practices conducted by Neuman (1985).

The safety benefits of left-turn channelization have been documented in several studies. A study by McFarland, Griffin, Rollins, Stockton, Phillips, and Dudek (1979) showed that accidents at signalized intersections where a left-turn lane was added, in combination with and without a left-turn signal phase, were reduced by 36 percent and 15 percent, respectively. At nonsignalized intersections with painted channelization separating the left-turn lane from the through lane, accidents were reduced for rural, suburban, and urban areas by 50, 30, and 15 percent, respectively. When raised channelization devices were used, the accident reductions were 60, 65, and 70 percent in rural, suburban, and urban areas, respectively. Hagenauer, Upchurch, Warren, and Rosenbaum (1982) found that the channelization of intersections reduced accidents by 32 percent and injury accidents by 50 percent.

On the other hand, it was reported in *Transportation Research Circular 382* (Transportation Research Board, 1991) that the aging driver, having poorer vision, slower physical reaction time, lower degree of awareness, and reduced ability to maneuver the vehicle, is more likely to be *negatively* affected by a raised median than is the average driver; and because medians are fixed objects, when they are struck they pose a serious threat of loss of control, especially for aging drivers. The typical curbed median offers low to no contrast with the adjacent pavement and is difficult to reflectorize at night. Low-beam headlight limitations, coupled with reduced vision of the aging driver, compounds the visibility problem. In addition, raised medians and raised corner islands, when used together, often create turning path options at complex intersections that are confusing to the average driver, and disproportionately so for the aging one. Thus, to realize the safety benefits channelization can provide, it is particularly important to ensure the visibility of raised surfaces for (older) drivers with diminished vision, so these road users can detect the channelizing devices and select their paths accordingly.

Another benefit in the use of channelization is the provision of a refuge for pedestrians. Refuge islands are a design element that can aid older pedestrians who have slow walking speeds. While the intent and purpose of the refuge island is well defined, no quantitative warrants are provided by either the MUTCD or AASHTO to determine when such an island is needed. However, areas where they are likely to be needed (e.g., multilane roadways and large or irregularly shaped intersections) are identified in both documents. Once the need is determined, the size and location of such islands can be determined with the help of these two documents. Also quite useful as a reference in this area is *Accessibility for Elderly and Handicapped Pedestrians—A Manual for Cities* (Earnhart and Simon, 1987).

With respect to the Hagenauer et al. (1982) study cited earlier, Hauer (1988) stated that because channelization in general serves to simplify an otherwise ambiguous and complex situation, the channelization of an existing intersection might enhance both the safety and mobility of older persons, as well as enhance the safety of other pedestrians and drivers. However, in designing a new intersection, he stated that the presence of islands is unlikely to offset the disadvantage of large intersection size for the pedestrian.

Staplin, Harkey, Lococo, and Tarawneh (1997) conducted a field study evaluating four right-turn lane geometries to examine the effect of channelized right-turn lanes and the presence of skew on right-turn maneuvers made by drivers of different ages. One hundred subjects divided across three age groups drove their own vehicles around test routes using the local street network in Arlington, VA. The three age groups were young/middle-aged (ages 25–45), young-old (ages 65–74), and old-old (age 75 and older). As diagrammed in figure 2, the four right-turn lane geometries were:

(a) A nonchannelized 90-degree intersection where drivers had the chance to make a right turn on red (RTOR) around a 12.2-m (40-ft) radius. This site served as a control geometry to examine how channelized intersections compare with nonchannelized intersections.

(b) A channelized right-turn lane at a 90-degree intersection with an exclusive use (acceleration) lane on the receiving street. Under this geometric configuration, drivers did not need to stop at the intersection and they were removed from the conflicting traffic upon entering the cross street. They had the opportunity to accelerate in their own lane on the cross street and then change lanes downstream when they perceived that it was safe to do so.

(c) A channelized right-turn lane at a 65-degree skewed intersection without an exclusive use lane on the receiving street.

(d) A channelized right-turn lane at a 90-degree intersection without an exclusive use lane on the receiving street. Under this geometry, drivers needed to check the conflicting traffic and complete their turn into a through traffic lane on the cross street.

The right-turn maneuver at all locations was made against two lanes carrying through (conflicting) traffic. The two through lanes were the only ones that had a direct effect on the right-turn maneuver. All intersections were located on major or minor arterials within a growing urban area, where the posted speed limit was 56 km/h (35 mi/h). All intersections were controlled by traffic signals with yield control on the three channelized intersections.

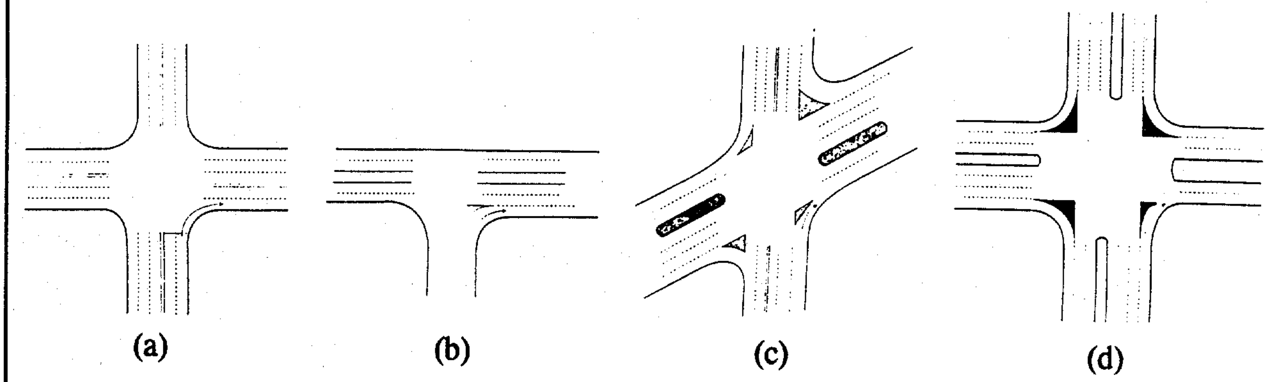


Figure 2. Intersection geometries examined in the Staplin et al. (1997) field study of right-turn channelization.

The results indicated that right-turn channelization affects the speed at which drivers make right turns and the likelihood that they will stop before making an RTOR. Drivers, especially younger drivers (ages 25–45), turned right at speeds 4.8–8 km/h (3–5 mi/h) higher on intersection approaches with channelized right-turn lanes than they did on approaches with nonchannelized right-turn lanes.

At the nonchannelized intersection, 22 percent of the young/middle-aged drivers, 5 percent of the young-old drivers, and none of the old-old drivers performed an RTOR without a stop. On approaches with channelized right-turn lanes, young/middle-aged and young-old drivers were much less likely to stop before making an RTOR. Where an acceleration lane was available, 65 percent of the young/middle-aged drivers continued through without a complete stop, compared with 55 percent of the young-old drivers and 11 percent of the old-old drivers. Old-old female drivers *always* stopped before an RTOR. The increased mobility exhibited by the two younger groups of drivers at the channelized right-turn lane locations was not, however, exhibited by the old-old drivers (age 75 and older), who stopped in 19 of the 20 turns executed at the channelized locations. Also, questionnaire results indicated drivers perceived that making a right turn on an approach with a channelized right-turn lane *without an acceleration lane on the cross street* was more difficult than at other locations, and even more difficult than at skewed intersections.

D. Design Element: Intersection Sight Distance (Sight Triangle)

Table 4. Cross-references of related entries for intersection sight distance.

Applications in Standard Reference Manuals		
AASHTO Green Book (1994)	Roadway Lighting Handbook Chapter 6 (1983)	NCHRP 279 Intersection Channelization Design Guide (1985)
Pg. 126-127, Sect. on *Decision Sight Distance* Pg. 696-721, Sect. on *Sight Distance* Pg. 796, Para. 5 through Pg. 801 Pgs. 938-939, Sect. on *Ramp Terminal Design*	Pg. 18, Form 2	Pg. 1, Item 1, 1st Bullet Pgs. 13-14, Sect. on *Corner Sight Distance* and Fig. 2-11 Pg. 35, Bottom Right Fig.

Because at-grade intersections define locations with the highest probability of conflict between vehicles, adequate sight distance is particularly important. Not surprisingly, a number of studies have shown that sight distance problems at intersections usually result in a higher accident rate (Mitchell, 1972; Hanna, Flynn, and Tyler, 1976; David and Norman, 1979). The need for adequate sight distance at an intersection is best illustrated by a quote from the Green Book: "The operator of a vehicle approaching an intersection at-grade should have an unobstructed view of the entire intersection and sufficient lengths of the intersecting highway to permit control of the vehicle to avoid collisions" (AASHTO, 1994). AASHTO values (for both uncontrolled and stop-controlled intersections) for available sight distance are measured from the driver's eye height (currently 1,070 mm [3.25 ft]) to the roofline of the conflicting vehicle (currently 1,300 mm [4.25 ft]).

Sight distances at an intersection can be reduced by a number of deficiencies, including physical obstructions too close to the intersection, severe grades, and poor horizontal alignment. The alignment and profile of an intersection have an impact on the sight distance available to the driver and thus affect the ability of the driver to perceive the actions taking place both at the intersection and on its approaches. Since proper perception is the first key to performing a safe maneuver at an intersection, it follows that sight distance should be maximized; this, in turn, means that the horizontal alignment should be straight and the gradients as flat as practical. Horizontal curvature on the approaches to an intersection makes it difficult for drivers to determine appropriate travel paths, because their visual focus is directed along lines tangential to these paths. Kihlberg and Tharp (1968) showed that accident rates increased 35 percent for highway segments with curved intersections over highway segments with straight intersections. Limits for vertical alignment at intersections suggested by AASHTO (1994) and Institute of Transportation Engineers (1984) are 3 and 2 percent, respectively.

Harwood, Mason, Pietrucha, Brydia, Hostetter, and Gittings (1993) stated that the provision of intersection sight distance (ISD) is intended to give drivers an opportunity to obtain the information they need to make decisions about whether to proceed, slow, or stop in situations where potentially conflicting vehicles may be present. They noted that while it is desirable to provide a reasonable margin of safety to accommodate incorrect or delayed driver decisions, there are substantial costs associated with providing sight distances at intersections; therefore, it is important that ISD requirements not be overly conservative or attempt to address traffic situations that are infrequent or unusual and for which increased ISD would provide little safety benefit.

Driver age differences in cognitive and physical capabilities that are relevant to ISD issues are discussed below. This is followed by a discussion of research efforts that have attempted to quantify the safety impact of providing adequate sight distance, plus studies that have examined the adequacy of the components that must be taken into account when calculating sight distance.

Older road users do not necessarily react more slowly to events that are expected, but they take significantly longer to make decisions about the appropriate response than younger road users, and this difference becomes more exaggerated in complex situations. Although the cognitive aspects of safe intersection negotiation depend upon a host of specific functional capabilities, the net result is response slowing. There is general consensus among investigators that older adults tend to process information more slowly than younger adults, and that this slowing not only transcends the slower reaction times often observed in older adults but may, in part, explain them (Anders, Fozard, and Lillyquist, 1972; Eriksen, Hamlin, and Daye, 1973; Waugh, Thomas, and Fozard, 1978; Salthouse and Somberg, 1982; Byrd, 1984). Of course, a conflict must be *seen* before any cognitive processing of this sort proceeds. Therefore, any decrease in available response time because of sight distance restrictions will pose disproportionate risks to older drivers. Slower reaction times for older versus younger adults when response uncertainty is increased has been demonstrated by Simon and Pouraghabagher (1978), indicating a disproportionately heightened degree of risk when older road users are faced with two or more choices of action. Also, research has shown that older persons have greater difficulty in situations where planned actions must be rapidly altered (Stelmach, Goggin, and Amrhein, 1988). The difficulty older persons experience in making extensive and repeated head movements further increases the decision and response times of older drivers at intersections.

David and Norman (1979) quantified the relationship between available sight distance and the expected reduction in accidents at intersections. The results of this study showed that intersections with shorter sight distances generally have higher accident rates. Using these results, predicted accident reduction frequencies related to ISD were derived as shown in table 5.

Table 5. Expected reduction in number of accidents per intersection per year. Source: David and Norman, 1979.

AADT* (1000s)	Increased Sight Distance (ft)		
	20-49	50-99	> 100
< 5	0.18	0.20	0.30
5 - 10	1.00	1.30	1.40
10 - 15	0.87	2.26	3.46
> 15	5.25	7.41	11.26

** annual average daily traffic entering the intersection*

Other studies have attempted to show the benefits to be gained from improvements to ISD (Mitchell, 1972; Strate, 1980). Mitchell conducted a before-and-after analysis, with a period of 1 year on each end, of intersections where a variety of improvements were implemented. The results showed a 67 percent reduction (from 39 to 13) in accidents where obstructions that inhibited sight distance were removed; this was the most effective of the implemented improvements. Strate's analysis examined 34 types of improvements made in Federal Highway Safety Program projects. The results indicated that sight distance improvements were the most cost-effective, producing a benefit/cost ratio of 5.33:1.

Collectively, the studies described above indicate a positive relationship between available ISD and a reduction in accidents, though the amount of accident reduction that can be expected by a given increase in sight distance may be expected to vary according to the maneuver scenario and existing traffic control at the intersection. Procedures for determining appropriate ISDs are provided by AASHTO for various levels of intersection control and the maneuvers to be performed. The scenarios defined are as follows:

- Case I: No Control. ISD for vehicles approaching intersections with no control, at which vehicles are not required to stop, but may be required to adjust speed.
- Case II: Yield Control. ISD for vehicles on a minor-road approach controlled by a yield sign.
- Case IIIA: Stop Control—Crossing Maneuver. ISD for a vehicle on a stop-controlled approach on the minor road to accelerate from a stopped position and cross the major road.
- Case IIIB: Stop Control—Left Turn. ISD for a vehicle on a stop-controlled approach on the minor road to accelerate from a stopped position and turn left onto the major road.
- Case IIIC: Stop Control—Right Turn. ISD for a vehicle on a stop-controlled approach on the minor road to accelerate from a stopped position and turn right onto the major road.
- Case IV: Signal Control (should be designed by Case III conditions). ISD for a vehicle on a signal-controlled approach.
- Case V: Stop Control—Vehicle Turning Left From Major Highway. ISD for a vehicle stopped on a minor road, waiting to turn left across opposing lanes of travel.

One of the principal components in determining ISD in all of these cases is perception-reaction time (PRT). The discussion of this value is first presented in chapters 2 and 3 of the Green Book under "Reaction Time" and "Brake Reaction Time," respectively (AASHTO, 1994). Results of several studies (e.g., Normann, 1953; Johansson and Rumar, 1971) are cited, and in conclusion, the 2.5-s value is selected since it was found to be adequate for approximately 90 percent of the overall driver population.

With respect to at-grade intersections, AASHTO recommends the following values of PRT for ISD calculations. In Case I, the PRT is assumed to be 2.0 s plus an additional 1.0 s to actuate braking, although the "preferred design" uses stopping sight distance (SSD) as the ISD design value (which incorporates a PRT of 2.5 s). In Case II, SSD is the design value; thus, the PRT is 2.5 s. For all Case III scenarios and Cases IV and V, the PRT is assumed to be 2.0 s.

A critique of these values questioned the basis for reducing the PRT from 2.5 s used in SSD calculations to 2.0 s in the Case III ISD calculations (Alexander, 1989). As noted by the author, "The elements of PRT are: detection, recognition, decision, and action initiation." For SSD, this is the time from object or hazard detection to initiation of the braking maneuver. Time to search for a hazard or object is not included in the SSD computation, and the corresponding PRT value is 2.5 s. Yet, in all Case III scenarios, the PRT has been reduced to 2.0 s and now includes a search component which was not included in the SSD computations. Alexander pointed out that a driver is looking straight ahead when deciding to perform a stopping maneuver and only has to consider what is in his/her forward view. At an intersection, however, the driver must look forward, to the right, and to the left. This obviously takes time, especially for those drivers with lower levels of physical dexterity, e.g., older drivers. Alexander (1989) proposed the addition of a "search time" variable to the current equations for determining ISD, and use of the PRT value currently employed in the SSD computations (i.e., 2.5 s) for all ISD computations. Neuman (1989) also argued that a PRT of 2.5 s for SSD may not be sufficient in all situations, and can vary from 1.5 s to 5.0 s depending on the physical state of the driver (alert versus fatigued), the complexity of the driving task, and the location and functional class of the highway.

A number of research efforts have been conducted to determine appropriate PRT values for use in ISD computations. Hostetter, McGee, Crowley, Seguin, and Dauber (1986) examined the PRT of 124 subjects traversing a 3-hour test circuit which contained scenarios identified above as Cases II, IIIA, IIIB, and IIIC. For the Case II (yield control) scenario, the results showed that in over 90 percent of the trials, subjects reacted in time to meet the SSD criteria established and thus the 2.5-s PRT value was adequate. With respect to Case III scenarios, the PRT was measured from the first head movement after a stop to the application of the accelerator to enter the intersection. The mean and 85th percentile values for all maneuvers combined were 1.82 s and 2.7 s, respectively. The results also showed that the through movement produced a lower value than the mean, while the turning maneuvers produced a higher value. These results lead to conclusions that the 2.0-s criteria for Case IIIA be retained and that the PRT value for the Case III turning maneuvers (B and C) be increased from 2.0 to 2.5 s. One other result, which is applicable to the current effort, was that no significant differences were found with respect to age, i.e., increased PRTs were needed to accommodate all drivers.

Another effort examined the appropriateness of the PRT values currently specified by AASHTO for computing SSD, vehicle clearance interval, sight distance on horizontal curves, and ISD (McGee and Hooper, 1983). With respect to ISD, the results showed the following: for Case I, the driver is not provided with sufficient time or distance to take evasive action if an opposing vehicle is encountered; and for Case II, adequate sight distance to stop before arriving at the intersection is not provided despite the intent of the standard to enable such action. With respect to the PRT values, recommendations include increasing the 2.0-s and 2.5-s values used in Case I and Case II calculations, respectively, to 3.4 s. It was also recommended that the PRT value for Case III scenarios be redefined.

Although there is no consensus from the above studies on the actual values of PRT that should be employed in the ISD computations, there is a very clear concern as to whether the current values are meeting the needs of older drivers. Since older drivers tend to take longer in making a decision, especially in complex situations, the need to further evaluate current PRT values is underscored. Slowed visual scanning of traffic on the intersecting roadway by older drivers has been cited as a cause of near misses of (crossing) accidents at intersections during on-road evaluations. In the practice of coming to a stop, followed by a look to the left, then to the right, and then back to the left again, the older driver's slowed scanning behavior allows approaching vehicles to have closed the gap by the time a crossing maneuver finally is initiated. The traffic situation has changed when the older driver actually begins the maneuver, and drivers on the main roadway are often forced to adjust their speed to avoid a collision. Hauer (1988) stated that "the standards and design procedures for intersection sight triangles should be modified because there is reason to believe that when a passenger car is taken as the design vehicle, the sight distance is too short for many older drivers, who take longer to make decisions, move their heads more slowly, and wish to wait for longer gaps in traffic."

In contrast, recent research conducted by Lerner, Huey, McGee, and Sullivan (1995) concluded that, based on older driver performance, no changes to design PRT values were recommended for ISD, SSD, or decision sight distance (DSD), even though the 85th percentile J values exceeded the AASHTO 2.0-s design standard at 7 of the 14 sites. The J value equals the sum of the PRT time and the time to set the vehicle in motion, in seconds. No change was recommended because the experimental design represented a worst-case scenario for visual search and detection (drivers were required to begin their search only after they had stopped at the intersection and looked inside the vehicle to perform a secondary task).

Lerner et al. (1995) conducted an on-road experiment to investigate whether the assumed values for Case III driver PRT used in AASHTO design equations adequately represent the range of actual PRT for older drivers. Approximately 33 subjects in each of three driver age groups were studied: ages 20–40, ages 65–69, and age 70 and older. Drivers operated their own vehicles on actual roadways, were not informed that their response times were being measured, and were naive as to the purpose of the study (i.e., they were advised that the purpose of the experiment was to judge road quality and how this relates to aspects of driving). The study included 14 data collection sites on a 90-km (56-mi) route. Results showed that the older drivers did *not* have longer PRT than younger drivers, and in fact the 85th percentile PRT closely matched the AASHTO design equation value of 2.0 s. The 90th percentile PRT was 2.3 s, with outlying values of 3–4 s. The median daytime PRT was approximately 1.3 s. Interestingly, it was found that typical driver actions did not follow the stop/search/decide maneuver sequence implied by the model; in fact, drivers continued to search and appeared ready to terminate or modify their maneuver even after they had begun to move into the intersection. This finding resulted in the study authors' conclusion that the behavioral model on which ISD is based is conservative.

Harwood, Mason, Brydia, Pietrucha, and Gittings (1996) evaluated current AASHTO policy on ISD for Cases I, II, III, IV, and V during performance of NCHRP project 15-14(1), based on a survey of current highway agencies' practices and a consideration of alternative ISD models and computational methodologies, as well as findings from observational studies for selected cases. Although this work culminated in recommendations for minimum distances for the major and minor legs of the sight triangle for all cases, driver age was not included as a study variable; therefore, specific values for these design elements were not included within the recommendations presented in this *Handbook*, nor is an exhaustive discussion of these materials included in this section. The results of the Harwood et al. (1996) analyses pertaining to ISD for Case IIIB and IIIC—and by extension for Case V—are of particular interest, however, in the interpretation of other, related findings from an older driver field study in this area. These analysis outcomes are reviewed below.

Prior to the 1990 AASHTO Green Book, the issue of ISD for a driver turning left off of a major roadway onto a minor roadway or into an entrance (Case V) was not specifically addressed. In the 1990 Green Book, the issue was addressed at the end of the Case III discussions in two paragraphs. In the 1994 Green Book, these same paragraphs have been placed under a new condition referred to as Case V. The equation used for determining ISD for Case V was simply taken from the Case IIIA (crossing maneuver at a stop-controlled intersection) and Case IIIB (left-turn maneuver from a stop-controlled minor road onto a major road) conditions, with the primary difference between the cases being the distance traveled during the maneuver. A central issue in defining the ISD for Case V involves a determination of whether the tasks that define ISD for Cases IIIA and IIIB are similar enough to the tasks associated with Case V to justify using the same equation, which follows:

$$ISD = 1.47\ V\ (J + t_a) \qquad \text{English}$$
$$ISD = 0.278\ V\ (J + t_a) \qquad \text{Metric}$$

where:

ISD = intersection sight distance (feet for English equation; meters for metric equation).
V = major roadway operating speed (mi/h for English equation; km/h for metric equation).
J = time required to search for oncoming vehicles, to perceive that there is sufficient time to make the left turn, and to shift gears, if necessary, prior to starting (J is currently assumed to be 2.0 s).
t_a = time required to accelerate and traverse the distance to clear traffic in the approaching lane(s); obtained from figure IX-33 in the AASHTO Green Book.

For Case IIIA (crossing maneuver), the sight distance is calculated based on the need to clear traffic on the intersecting roadway on both the left and right sides of the crossing vehicle. For Case IIIB (left turn from a stop), sight distance is based on the requirement to first clear traffic approaching from the left and then enter the traffic stream of vehicles from the right. It has been demonstrated that the perceptual judgments required of drivers in both of these maneuver situations increase in difficulty when opposing through traffic must be considered.

The perceptual task of turning left from a major roadway at an unsignalized intersection or during a permitted signal phase at a signalized intersection requires a driver to make time-distance estimates of a longitudinally moving target as opposed to a laterally moving target. Lateral movement (also referred to as tangential movement) describes a vehicle that is crossing an observer's line of sight, moving against a changing visual background where it passes in front of one fixed reference point after another. Longitudinal movement, or movement in depth, results when the vehicle is either coming toward or going away from the observer. In this case there is no change in visual direction, only subtle changes in the angular size of the visual image, typically viewed against a constant background. Longitudinal movement is a greater problem for drivers because the same displacement of a vehicle has a smaller visual effect than when it moves laterally—that is, lateral movement results in a much higher degree of relative motion (Hills, 1980).

In comparison with younger subjects, a significant decline for older subjects has been reported in angular motion sensitivity. In a study evaluating the simulated change in the separation of taillights indicating the overtaking of a vehicle, Lee (1976) found a threshold elevation greater than 100 percent for drivers ages 70–75 compared with drivers ages 20–29 for brief exposures at night. Older persons may in fact require twice the rate of movement to perceive that an object's motion in depth is approaching, versus maintaining a constant separation or receding, given a brief duration (2.0 s) of exposure. In related experiments, Hills (1975) found that older drivers required significantly longer to perceive that a vehicle was moving closer at constant speed: at 31 km/h (19 mi/h), decision times increased 0.5 s between ages 20 and 75. This body of evidence suggests that the 2.0-s PRT (i.e., variable J in the ISD equation above) used for Cases III and V may not be sufficient for the task of judging gaps in opposing through traffic by older drivers. A revision of Case V to determine a minimum required sight distance value which more accurately reflects the perceptual requirements of the left-turn task may therefore be appropriate.

Harwood et al. (1996) suggested that at locations where left turns from the major road are permitted at intersections and driveways, at unsignalized intersections, and at signalized intersections without a protected turn phase, sight distance along the major road should be provided based on a critical gap approach, as was recommended for left and right turns from the minor road at stop-controlled intersections. The gap acceptance model developed and proposed to replace the current ISD AASHTO model is:

$$ISD = 1.47\ VG \qquad \text{English}$$
$$ISD = 0.278\ VG \qquad \text{Metric}$$

where:

ISD = intersection sight distance (feet for English equation; meters for metric equation).

V = operating speed on the major road (mi/h for English equation, km/h for metric equation).

G = the specified critical gap (in seconds); equal to 5.5 s for crossing one opposing lane plus an additional 0.5 s for each additional opposing lane.

Field data were collected in the NCHRP study to better quantify the gap acceptance behavior of passenger car and truck drivers, but only for left- and right-turning maneuvers from minor roadways controlled by a STOP sign (Cases IIIB and C). In the Phase I interim report produced during the conduct of the NCHRP project, Harwood et al. (1993) reported that the critical gap currently used by the California Department of Transportation is 7.5 s. When current AASHTO Case IIIB ISD criteria are translated to time gaps in the major road traffic stream, the gaps range from 7.5 s (67 m [220 ft]) at a 32-km/h (20-mi/h) operating speed to 15.2 s (475 m [1,560 ft]) at a 112-km/h (70-mi/h) operating speed. Harwood et al. (1993) stated that the rationale for gap acceptance as an ISD criterion is that drivers safely accept gaps much shorter than 15.2 s routinely, even on higher speed roadways.

In developing the gap acceptance model for Case V, Harwood et al. (1996) relied on data from studies conducted by Kyte (1995) and Micsky (1993). Kyte (1995) recommended a critical gap value of 4.2 s for left turns from the major road by passenger cars for inclusion in the unsignalized intersection analysis procedures presented in the *Highway Capacity Manual* (Transportation Research Board, 1994). A constant value was recommended regardless of the number of lanes to be crossed; however, a heavy-vehicle adjustment of 1.0 s for two-lane highways and 2.0 s for multilane highways was recommended. Harwood et al. (1996) reported that Micsky's 1993 evaluation of gap acceptance behavior for left turns from the major roadway at two Pennsylvania intersections resulted in critical gaps with a 50 percent probability of acceptance (determined from logistic regression) of 4.6 s and 5.3 s. Using the rationale that design policies should be more conservative than operational criteria such as the *Highway Capacity Manual*, Harwood et al. (1996) recommended a critical gap for left turns from the major roadway of 5.5 s, and an increase in the critical gap to 6.5 s for left turns by single-unit trucks and to 7.5 s for left turns by combination trucks. In addition, if the number of opposing lanes to be crossed exceeds one, an additional 0.5 s per additional lane for passenger cars and 0.7 s per additional lane for trucks was recommended.

It is important to note that the NCHRP study did not consider driver age as a variable. However, Lerner et al. (1995) collected judgments about the acceptability of gaps in traffic as a function of driver age for left turn, right turn, and through movements at stop-controlled intersections. While noting that these authors found no significant differences between age groups in the *total* time required to perceive, react, and complete a maneuver in a related Case III PRT study, the Lerner et al. (1995) findings indicate that younger drivers accept shorter gaps than older drivers. The 50th percentile gap acceptance point was about 7 s (i.e., if a gap is 7 s long, only about half of the subjects would accept it). The 85th percentile point was approximately 11 s. The oldest group required about 1.1 s longer than the youngest group.

In an recently completed observational field study of driver performance as a function of left-turn lane geometry, mean left-turn critical gap sizes (in seconds) across four locations where the main road operating speed was 56 km/h (35 mi/h), for drivers who had positioned their vehicles within the intersection, were 5.90 s (young/middle-aged [ages 25–45] females), 5.91 s (young/middle-aged [ages 25–45] males), 6.01 s (young-old [ages 65–74] females), 5.84 s (young-old [ages 65–74] males), 6.71 s (old-old [age 75 and older] females), and 6.55 s (old-old [age 75 and older] males). Prominent trends indicated that older drivers demonstrated larger critical gap values at all locations. The young/middle-aged and young-old groups were not significantly different from each other; however, both were significantly different from the old-old group. Critical gap sizes displayed in a laboratory simulation study in the same project, where oncoming vehicles traveling at 56 km/h (35 mi/h) were viewed on a large screen display in correct perspective, ranged from 6.4 s to 8.1 s for young/middle-aged drivers and from 5.8 to 10.0 s for drivers age 75 and older (Staplin, Harkey, Lococo, and Tarawneh, 1997).

These diverse findings argue that an appropriate value for G in the gap acceptance model will lie toward the upper end of the 7- to 11-s range to accommodate older drivers, while also preserving a margin of safety. This strategy acknowledges the diminished capability of older drivers to judge oncoming vehicle speed in a situation that places this group of road users at particular risk, i.e., when an opposing through vehicle approaches at excessive speed.

Regarding PRT for Cases III and V, AASHTO (1994) assumes a PRT of 2.0 s as the time necessary for the driver to look in both directions of the roadway, to perceive that there is sufficient time to perform the maneuver safely, and to shift gears, if necessary, prior to starting. This value is based on research performed by Johansson and Rumar (1971). The PRT is defined as the time from the driver's first look for possible oncoming traffic to the instant the car begins to move. Some of these operations are done simultaneously by many drivers, and some operations, such as shifting gears, may be done before searching for intersecting traffic or may not be required with automatic transmissions. AASHTO states that a value of 2.0 s is assumed to represent the time taken by the slower driver.

Regarding the value of t_a, which is read from figure IX-33 in the AASHTO Green Book, the Staplin et al. (1997) data found no significant differences in maneuver time as a function of age for the drivers turning left at the four intersections studied (which had distances ranging from 26 to 32 m [84 to 106 ft]). Maneuver times for drivers positioned within the intersection versus unpositioned drivers, however, were significantly different. Since significantly fewer older drivers positioned themselves in the field study, the design value for this factor (maneuver time) should be based on that obtained for unpositioned drivers. A comparison of the 95th percentile clearance times demonstrated by positioned drivers and unpositioned drivers at each location with AASHTO values is presented in table 6.

Current and proposed sight distance models were exercised by Staplin et al. (1997) using data collected in the observational field study. For this comparison, two basic models were selected. Model 1 was the current model in the AASHTO Green Book for computing ISD, which relied on a PRT of 2.0 s and maneuver time taken from figure IX-33 in the Green Book. Model 2 was the gap acceptance model developed as part of NCHRP project 15-14(1), which relies on the critical gap in place of PRT and maneuver time. Model 2 takes the form shown below, with all terms as defined on page 56 in this section:

$$ISD = 1.47\ VG \qquad \text{English}$$
$$ISD = 0.278\ VG \qquad \text{Metric}$$

Each of these models was used with the appropriate design values to compute the required sight distance at each of the selected intersections. The models were then used with adjusted design values or actual data collected in the field to also determine the required sight distance at each location.

Table 6. Comparison of clearance times obtained in the Staplin et al. (1997) field study with AASHTO Green Book values used in sight distance calculations.

Measure	Vehicle Location	Left-Turn Lane Geometry			
		-14-ft Offset	-3-ft Offset	0-ft Offset	+ 6-ft Offset
Distance Traveled (ft)	Positioned	70 ft	67 ft	64 ft	70 ft
95th Percentile Clearance Time (s) From Field Study	Positioned	3.8 s	3.9 s	3.9 s	3.9 s
AASHTO Clearance Time (s) From Figure IX-33	Positioned	5.1 s	5.0 s	5.0 s	5.1 s
Distance Traveled (ft)	Unpositioned	106 ft	98 ft	84 ft	88 ft
95th Percentile Clearance Time (s) From Field Study	Unpositioned	6.7 s	6.4 s	6.6 s	5.7 s
AASHTO Clearance Time (s) From Figure IX-33	Unpositioned	6.5 s	6.2 s	5.9 s	6.0 s

1 ft = 0.305 m

The first adjustment made to the current AASHTO model was an increase in the PRT. As previously noted, several studies have examined and critiqued the use of 2.0 s for PRT in this model. Thus, an adjusted model (Model 3) with a PRT of 2.5 s, which is equivalent to the value used in SSD calculations, was also included in the analysis.

One of the data elements collected as part of this research was the maneuver time of the left-turning driver. This time is equivalent to t_a in the AASHTO model. These times were measured from two locations, depending on whether or not the drivers positioned themselves within the intersection prior to turning. The first location was from a position within the intersection, approaching the median or centerline of the cross street. The second location was at or behind the stop bar or end of the left-turn bay. Using the original AASHTO model and these field data maneuver times, sight distances were computed with two additional models, substituting the 95th percentile maneuver time from the distribution of all unpositioned drivers in one model (Model 4) and the 95th percentile maneuver time from the distribution of all positioned drivers in the other model (Model 5).

Critical gap data were also collected and analyzed by driver age group, at each of the intersections studied. The drivers age 75 and older accepted significantly larger gaps compared with the other age groups. Thus, two different critical gaps were used in adjusted gap models to compute the required sight distances. These models simply modify the value of G in Model 2. In the first adjustment, the critical gap for all drivers (across age) as measured in the field was substituted for the value of G (Model 6); in the second adjustment, the critical gap for drivers age 75 and older as measured in the field was substituted for the value of G (Model 7).

A detailed discussion of the outputs from the model exercise is provided in the publication *Intersection Geometric Design and Operational Guidelines for Older Drivers and Pedestrians* (Staplin, Harkey, Lococo, and Tarawneh, 1997). However, one significant result is that the required sight distances computed using the modified AASHTO model (where PRT was increased to 2.5 s) produced required sight distance values that were the most predictive of actual field operations. Thus, if the current AASHTO model is deemed to be the appropriate one for calculating ISD as it relates to drivers turning left from a major roadway, there is evidence that the PRT value should be increased to 2.5 s to provide adequate sight distance at most locations. The gap acceptance model, on the other hand, produced sight distance values that were approximately 23 percent shorter than the current AASHTO model. If the gap acceptance model is going to be used, particularly where there are significant volumes of older left-turning drivers, there may need to be an adjustment factor applied to increase the sight distance to better accommodate this driver age group. Also, to the extent that the current AASHTO ISD model produces longer sight distances than the gap acceptance model, it may be most prudent—considering the increasing range of driver (diminished) capabilities—to regard the difference as simply an additional margin of safety.

E. Design Element: Opposite (Single) Left-Turn Lane Geometry, Signing, and Delineation

Table 7. Cross-references of related entries for opposite (single) left-turn lane geometry, signing, and delineation.

Applications in Standard Reference Manuals		
MUTCD (1988)	AASHTO Green Book (1994)	NCHRP 279 Intersection Channelization Design Guide (1985)
Pg. 2A-12, Sect. 2A-31 Pg 2E-22 to 2E-23 (Sect 2E-40) Pg. 3A-2, Para. 6, Item 1,2,4 Pg. 3A-4, Sect 3A-10 Pg 3A-4, Sect 3A-7, item 8 Pg. 3B-11, Para. 3-5 Pg. 3B-21, Para. 4 Pg.3B-22, Sect. 3B-17 Pg. 3B-27, Para 2 Pg. 3B-29, Fig 3-18 Pg. 5F-1, Sect 5F-5	Pg. 45, Para. 1 Pgs. 679-689, Sect(s). on *Island Size and Designation,* Pg 783, Para. 4 Pgs. 786-787, Sect. on *Median End Treatment* Pg. 787, Para. 1	Pg. 34, Para. 1 and Top Fig.

Studies examining older driver crashes and the types of maneuvers being performed just prior to the collision have consistently found this group to be overinvolved in left-turning accidents at both rural and urban signalized intersections and have indicated that failure to yield the right-of-way (as the turning driver) was the principal violation type (Staplin and Lyles, 1991; Council and Zegeer, 1992). Underlying problems for the maneuver errors include the misjudgment of oncoming vehicle speed, misjudgment of available gap, assuming the oncoming vehicle was going to stop or turn, and simply not seeing the other vehicle. Joshua and Saka (1992) noted that sight distance problems at intersections which result from queued vehicles in opposite left-turn lanes pose safety and capacity deficiencies, particularly for unprotected (permitted) left-turn movements. These researchers found a strong correlation between the offset for opposite left-turn lanes—i.e., the distance from the inner edge of a left-turn lane to the outer edge of the opposite left-turn lane—and the available sight distance for left-turning traffic.

The alignment of opposite left-turn lanes and the horizontal and vertical curvature on the approaches are the principal geometric design elements that determine how much sight distance is available to a left-turning driver. Operationally, vehicles in the opposite left-turn lane waiting to turn left can also restrict the (left-turning) driver's view of oncoming traffic in the through lanes. The level of blockage depends on how the opposite left-turn lanes are aligned with respect to each other, as well as the type/size of vehicles in the opposing queue. Restricted sight distance can be minimized or eliminated by offsetting opposite left-turn lanes so that left-turning drivers do not block each other's view of oncoming through traffic. When the two left-turn lanes are exactly aligned, the offset distance has a value of zero. Negative offset describes the situation where the opposite left-turn lane is shifted to the left. Positive offset describes the situation where the opposite left-turn lane is shifted to the right. Positively offset left-turn lanes and aligned left-turn lanes provide greater sight distances than negatively offset left-turn lanes, and a positive offset provides greater sight distance than the aligned configuration.

Older drivers may experience greater difficulties at intersections as the result of diminished visual capabilities such as depth and motion perception, as well as diminished attention-sharing (cognitive) capabilities. Studies have shown that there are age differences in depth and motion perception. Staplin, Lococo, and Sim (1993) found that the angle of stereopsis (seconds of arc) required for a group of drivers age 75 and older to discriminate depth using a commercial vision tester was roughly twice as large as that needed for an 18–55-year-old group to achieve the same level of performance. However, while accurate perception of the distance to geometric features delineated at intersections—as well as to potentially hazardous objects such as islands, pedestals, and other raised features—is important for the safe use of these facilities, relatively greater attention by researchers has been placed upon motion perception, where dynamic stimuli (usually other vehicles) are the primary targets of interest. It has been shown that older persons require up to twice the rate of movement to perceive that an object is approaching, and they require significantly longer to perceive that a vehicle is moving closer at a constant speed (Hills, 1975). A study investigating causes of older driver overinvolvement in turning accident at intersections, building on the previously reported decline for detection of angular expansion cues, did not find evidence of overestimation of time-to-collision (Staplin et al., 1993). At the same time, a relative insensitivity to approaching (conflict) vehicle speed was shown for older versus younger drivers; this result was interpreted as supporting the notion that older drivers rely primarily or exclusively on perceived *distance*—not time or velocity—to perform gap acceptance judgments, reflecting a reduced ability to integrate time and distance information with increasing age. Thus, a principal source of risk at intersections is the error of an older, turning driver in judging gaps in front of fast vehicles.

Several recent studies examining the minimum required sight distance for a driver turning left from a major roadway to a minor roadway, as a function of major road design speed, have provided data necessary to determine: (1) the left-turn lane offset value needed to achieve the minimum required sight distance and (2) the offset value that will provide unlimited sight distance. A fundamental premise in these studies, which are described below, is that it is *not* the amount of left-turn lane offset per se, but rather the sight distance which a given level of offset provides that should be the focus of any recommendations pertaining to the design of opposite left-turn lanes.

In a study conducted by McCoy, Navarro, and Witt (1992), guidelines were developed for offsetting opposite left-turn lanes to eliminate the left-turn sight distance problem. All minimum offsets specified in the guidelines are positive. For 90-degree intersections on level tangent sections of four-lane divided roadways, with 3.6-m (12-ft) left-turn lanes in 4.9-m (16-ft) medians with 1.2-m (4-ft) medial separators, the following conclusions were stated by McCoy et al.: (1) a 0.6-m (2-ft) positive offset provides unrestricted sight distance when the opposite left-turn vehicle is a passenger car, and (2) a 1.06-m (3.5-ft) positive offset provides unrestricted sight distance when the opposite left-turn vehicle is a truck, for design speeds up to 113 km/h (70 mi/h).

Harwood, Pietrucha, Wooldridge, Brydia, and Fitzpatrick (1995) conducted an observational field study and an accident analysis to develop design policy recommendations for the selection of median width at rural and suburban divided highway intersections based on operational and safety considerations. They found that at rural unsignalized intersections, both accidents and undesirable driving behaviors decrease as median width increases. However, at suburban signalized and unsignalized intersections, accidents and undesirable behaviors increase as the median width increases. At suburban intersections, it is therefore suggested that the median should not generally be wider than necessary to accommodate pedestrians and the appropriate median left-turn treatment needed to serve current and anticipated future traffic volumes. Harwood et al. stated that wider medians generally have positive effects on traffic operations and safety; however, wider medians can result in sight restrictions for left-turning vehicles due to the presence of opposite left-turn vehicles. The most common solution to this problem is to offset the left-turn lanes, using either parallel offset or tapered offset left-turn lanes.

Figure 3 compares conventional left-turn lanes with these two alternative designs. As noted by Harwood et al. (1995), parallel and tapered offset left-turn lanes are still not common, but are used increasingly to reduce the risk of accidents due to sight restrictions from opposite left-turn vehicles. Parallel offset left-turn lanes with 3.6-m (12-ft) widths can be constructed in raised medians with widths as narrow as 7.2 m (24 ft), and can be provided in narrower medians if restricted lane widths or curb offsets are used or a flush median is provided (Bonneson, McCoy, and Truby, 1993). Tapered offset left-turn lanes generally require raised medians of 7.2 m (24 ft) or more in width.

Staplin, Harkey, Lococo, and Tarawneh (1997) performed a laboratory study, field study, and sight distance analysis to measure driver age differences in performance under varying traffic and operating conditions, as a function of varying degrees of offset of opposite left-turn lanes at suburban arterial intersections. Research findings indicated that an increase in sight distance through positively offsetting left-turn lanes can be beneficial to left-turning drivers, particularly older drivers. In the field study, where left-turn vehicles needed to cross the paths of two or three lanes of conflicting traffic (excluding parking lanes) at 90-degree, four-legged intersections, four levels of offset of opposite left-turn lane geometry were examined. These levels include: (a) 1.8-m (6-ft) "partial positive" offset, (b) aligned (no offset) left-turn lanes, (c) 0.91-m (3-ft) "partial negative" offset, and (d) 4.3-m (14-ft) "full negative" offset. All intersections were located within a growing urban area where the posted speed limit was 56 km/h (35 mi/h). Additionally, all intersections were controlled by traffic-responsive semi-actuated signals, and all left-turn maneuvers were completed during the permitted left-turn phase at all study sites. In the analysis of the field study lateral positioning data, it was found that the partial positive offset and aligned locations had the same effect on the lateral positioning behavior of drivers. Drivers moved approximately 1.5 m (5 ft) to the left when there was a large negative offset, clearly indicating that sight distance was limited. There was a significant difference between the partial negative offset geometry and the partial positive offset or aligned geometries, suggesting a need for longer sight distance when opposite left-turn lanes are even partially negatively offset. The fact that older drivers (and females) were less likely to position themselves (i.e., pull into the intersection) in the field studies highlights the importance of providing adequate sight distance for unpositioned drivers, for all left-turn designs.

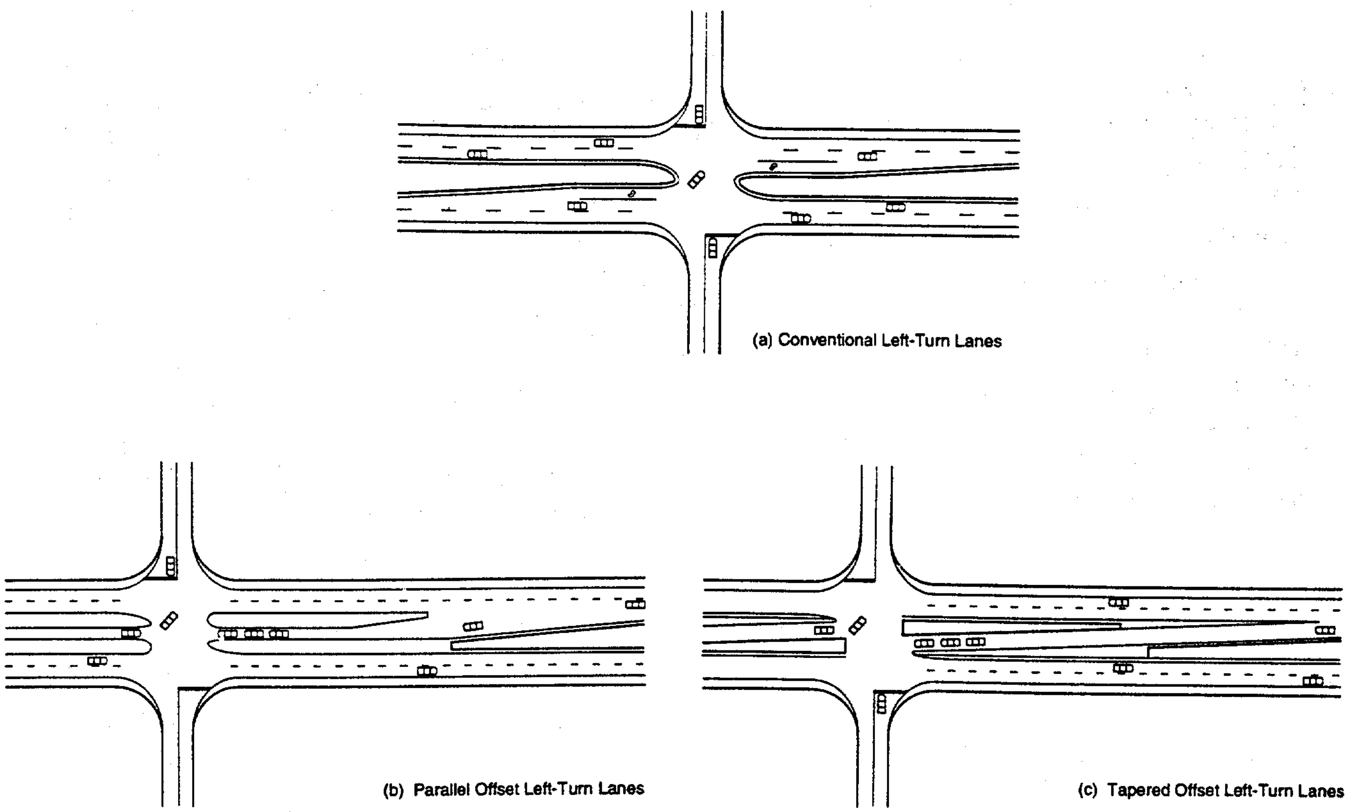
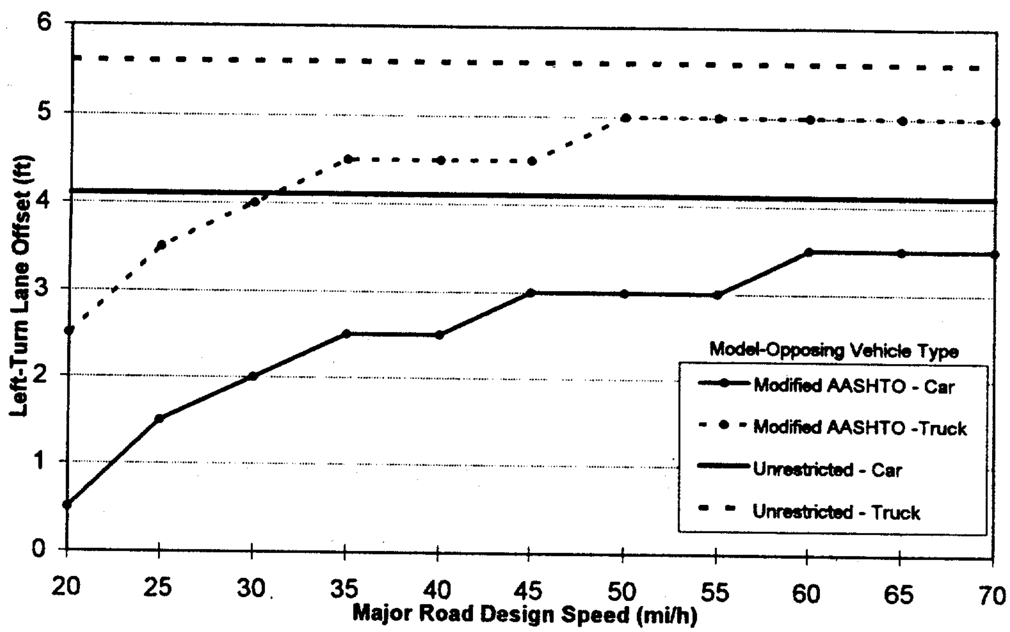


Figure 3. Alternative left-turn treatments for rural and suburban divided highways. Source: Bonneson, McCoy, and Truby (1993).

Several issues were raised in the research conducted by Staplin et al. (1997) regarding the adequacy of the current and proposed intersection sight distance (ISD) models for a driver turning left from a major roadway. The researchers exercised seven sight distance models, including the current AASHTO Case V model using 2.0 s for perception-reaction time (PRT), a modified AASHTO model using a 2.5-s PRT, and a gap acceptance model proposed by Harwood, Mason, Brydia, Pietrucha, and Gittings (1996). The NCHRP-proposed gap acceptance model relies on a critical gap value in place of PRT and maneuver time. A detailed description of the model parameters and output can be found in the FHWA report entitled *Intersection Geometric Design and Operational Guidelines for Older Drivers and Pedestrians* (Staplin et al., 1997). Of particular significance was the finding that the modified AASHTO model with the longer PRT of 2.5 s was the model most predictive of actual field operations. Also of significance was the dramatic decrease in required sight distance that occurred with the gap acceptance model compared with the traditional AASHTO model. Across all intersections and all design speeds, the required sight distance was approximately 23 percent less using the gap acceptance model. However, this was expected since the rationale behind the use of a gap acceptance model (cf. Harwood et al., 1996), in place of the current AASHTO model, is the fact that drivers are commonly observed accepting shorter gaps than those implied by the current model.

Regardless of which model is deemed most appropriate for computing ISD for drivers turning left off a major roadway, one way to increase the sight distance is through positively offset left-turn lanes. As shown in the study by Staplin et al. (1997), such designs result in significantly better performance on the part of all drivers, but especially for older drivers. Prior work by McCoy et al. (1992) examined the issue of offset left-turn lanes and developed an approach that could be used to compute the amount of offset that is required to minimize or eliminate the sight restriction caused by opposing left-turn vehicles. This approach was applied to the intersections in the study by Staplin et al. (1997) to determine the amount of offset that would be required when using the modified AASHTO model (i.e., J = 2.5 s). The left-turn lane offsets required to achieve the minimum required sight distances calculated using this model are shown in figure 4, in addition to the offsets required to provide unrestricted sight distance. Based on intersections examined in the study, the offset necessary to achieve unrestricted sight distance for opposing left-turning cars is 1.2 m (4.1 ft) and for opposing left-turning trucks is 1.7 m (5.6 ft).



1 ft = 0.305 m
1 mi/h = 1.61 km/h

Figure 4. Left-turn lane offset design values required to achieve minimum required sight distances using the modified AASHTO model (J=2.5 s) and unrestricted sight distances.

Finally, the potential for wrong-way maneuvers, particularly by older drivers, at intersections with positively offset channelized left-turn lanes was raised during a panel meeting comprised of older driver experts and highway design engineers, during the conduct of the research performed by Staplin et al. (1997). The concern expressed was that drivers turning left from the minor road may turn too soon and enter the channelized left-turn lane, instead of turning around both medians. Similar concern was raised by highway engineers surveyed by Harwood et al. (1995) during the conduct of NCHRP project 15-14(2). These authors reported that the potential for wrong-way movements by opposing-direction vehicles entering the left-turn roadway is minimal if proper signing and pavement markings are used. Researchers studying wrong-way movements at intersections—particularly the intersection of freeway exits with secondary roads—have found that such movements resulted from left-turning vehicles making an early left turn rather than turning around the nose of the median, and have proposed and tested several countermeasures. Parsonson and Marks (1979) found that the use of a wide (610 mm [24 in]) white stop bar was effective in preventing wrong-way entries onto freeway exit ramps in Georgia, as was the use of the two-piece, 7-m (23.5-ft) painted arrow pavement marking (wrong-way arrow). Scifres and Loutzenheiser (1975) reported that indistinct medians are design elements that reduce a driver's ability to see and understand the overall physical and operational features of an intersection, increasing the frequency of wrong-way movements. They suggested delineation of the median noses to increase their visibility and improve driver understanding of the intersection design and function.

F. Design Element: Edge Treatments/Delineation of Curbs, Medians, and Obstacles

Table 8. Cross-references of related entries for edge treatments/delineation of curbs, medians, and obstacles.

Applications in Standard Reference Manuals			
MUTCD (1988)	**AASHTO Green Book (1994)**	**Roadway Lighting Handbook Chapter 6 (1983)**	**NCHRP 279 Intersection Channelization Design Guide (1985)**
Pg. 3A-2, Para. 6, Item 1 Page 3A-4, Item 9. Pg. 3A-4, Sect. on *Curb Markings* Pg. 3B-13, Sect. on *Approach to an Obstruction* Pg. 3B-8, Sect. on *Pavement Edge Lines* Pg. 3B-14, Sect. on *Median Islands Formed by Pavement Markings* Pg. 3B-21, Para. 4 Page 3B-22, Sect. on *Substituting for Pavement Markings* Pgs. 3C-1–3C-4, Sect. on *Object Markings* Pg. 3D-1, Sect. on *Curb Markings for Delineation* Pg. 3D-2, Para. 3 Pg. 3D-3, Sect. on *Delineator Placement & Spacing* Pg. 5C-1, Sect. on *Approach End Treatment* Pg. 5F-1, Sect. on *Markings*	Pg. 45, Para. 1 Pg. 314, Para. 7 Pg. 315, Para. 2 Pg. 347, Para. 5 Pg. 348, Para(s) 1-3 Pg. 637, Para. 7 Pgs. 679-689, Sect(s). on *Island Size and Designation, Delineation and Approach-End Treatment,* and *Right-Angle Turns With Corner Islands* Pg. 755, Sect. on *Shape of Median End* Pg. 783, Para(s). 2-4 Pgs. 786-787, Sect. on *Median End Treatment* Pg. 927, Para(s) 1 & 3 Page 929, Para. 9 and Fig. X-68	Pg. 21, Table 1	Pg. 35, Para. 2

The discrimination at a distance of gross highway features, as opposed to the fine detail contained in a sign message, governs drivers' perceptions of intersection geometric elements. Thus, the conspicuity of such elements as curbs, medians, and obstacles, as well as all raised channelization, is of paramount importance in the task of safely approaching and choosing the correct lane for negotiating an intersection, as well as avoiding collisions with the raised surfaces. During the conduct of their driving task analysis, McKnight and Adams (1970a, 1970b) identified five driving tasks related specifically to the conspicuity of intersection geometric elements: (1) maintain correct lateral lane position, (2) survey pavement markings, (3) survey physical boundaries, (4) determine proper lane position for the intended downstream maneuver, and (5) search for path guidance cues. The visual/perceptual requirement common to the performance of these tasks is contrast sensitivity: for detecting lane lines, painted roadway symbols and characters, curbs and roadway edge features, and median barriers.

Older drivers' decreased contrast sensitivity, reduced useful field of view, increased decision time—particularly in response to unexpected events—and slower vehicle control movement execution combine to put these highway users at greater accident risk when approaching and negotiating intersections. The smaller the attentional demand required of a driver to maintain the correct lane position for an intended maneuver, the greater the attentional resources available for activities such as the recognition and processing of traffic control device messages and detection of conflict vehicles and pedestrians.

A variety of conspicuity-enhancing treatments are mandated in current practice. The MUTCD (section 3B-13) specifies that pavement markings shall be used to guide traffic on the approach to fixed objects (such as channelization islands) within a paved roadway. Section 3B-21 (Curb Markings for Parking Restrictions) states that curb markings of yellow and white are used for delineation and visibility; section 3D-3 (Curb Markings for Delineation) states that reflectorized solid yellow (delineators) should be placed on the curbs of islands located in the line of traffic flow where the curb serves to channel traffic to the right of the obstruction, and reflectorized solid white (delineators) should be used when traffic may pass on either side of the island. Supplementary treatments, and requirements for in-service brightness levels for certain elements contained in existing guidelines, are presently at issue.

The conspicuity of curbs and medians, besides aiding in the visual determination of how an intersection is laid out, is especially important when medians are used as pedestrian refuges. Care must be taken to ensure that pedestrian refuges are clearly signed and made as visible as possible to passing motorists.

Research findings describing driver performance differences directly affecting the use of pavement markings and delineation focus upon (age-related) deficits in spatial vision. In a pertinent laboratory study conducted by Staplin, Lococo, and Sim (1990), two groups of subjects (ages 19–49 and 65–80) viewing a series of ascending and descending brightness delineation targets were asked to report when they could just detect a roadway heading—left versus right—from simulated distances of 30.5 m (100 ft) and 61 m (200 ft). Results showed that the older driver group required a contrast of 20 percent higher than the younger driver group to achieve the discrimination task in this study.

To describe the magnitude of the effects of age and visual ability on delineation detection/recognition distance and retroreflective requirements for threshold detection of pavement markings, a series of analyses using the Ford Motor Company PC DETECT computer model (cf. Matle and Bhise, 1984) yielded the stripe contrast requirements shown in table 9. PC DETECT is a headlamp seeing-distance model which uses the Blackwell and Blackwell (1971, 1980) human contrast sensitivity formulations to calculate the distance at which various types of targets illuminated by headlamps first become visible to approaching drivers, with and without glare from opposing headlights. The top 5 percent of 25-year-olds (the best-performing younger drivers) and bottom 5 percent of 75-year-olds (the worst-performing older drivers) were compared in this analysis.

Blackwell and Taylor (1969) conducted a study of surface pavement markings employing an interactive driving simulator, plus field evaluations. They concluded that driver performance —measured by the probability of exceeding lane limits—was optimized when the perceived brightness contrast between pavement markings and the roadway was 2.0. A study by Allen, O'Hanlon, and McRuer (1977) also concluded that delineation contrast should be maintained above a value of 2.0 for adequate steering performance under clear night driving conditions. In other words, because contrast is defined as the difference between target and background

Table 9. Contrast requirements for edgeline visibility at 122 m (400 ft) with 5-s preview at a speed of 88 km/h (55 mi/h), as determined by PC DETECT computer model.

Driver age group/ % accommodated	Worst-case glare	No glare
Age 25 / top 5%	0.11	0.05
Age 75 / bottom 5%	7.21	3.74

luminance, divided by the background luminance alone, these studies have asserted that markings must appear to be *at least* three times as bright as the road surface. Also, because these studies were not specifically focused on the accommodation of older drivers—particularly the least capable members of this group—the contrast requirements defined in more recent modeling studies and analyses, as presented in table 9, are accorded greater emphasis. Taking the indicated value for the least capable 5 percent of 75-year-olds into account, as well as the prior field evaluations, a contrast requirement of 3.0 for pavement markings appears most reasonable.

Note that luminance may be measured in candelas per square meter (cd/m^2) or in footlamberts (fL), but contrast is a dimensionless number; thus the calculation of contrast level is independent of the unit of measure.

Finally, inadequate conspicuity of raised geometric features at intersections has been brought to the attention of researchers during the conduct of several focus group studies involving older drivers. Subjects reported difficulty knowing where to drive, due to missing or faded roadway lines on roadway edges and delineation of islands and turning lanes. They also reported hesitating during turns, because they did not know where to aim the vehicle (Staplin, Lococo, and Sim, 1990). In another focus group, subjects suggested that the placement of advance warning pavement markings be located as far in advance of an intersection as practicable (Council and Zegeer, 1992). Drivers ages 66–77 and older participating in focus group discussions conducted by Benekohal, Resende, Shim, Michaels, and Weeks (1992), reported that intersections with too many islands are confusing because it is hard to find which island the driver is supposed to go around. Raised curbs that are unpainted are difficult to see, especially in terms of height and direction, and result in people running over them. These older drivers stated that they would prefer to have rumble strips in the pavement to warn them of upcoming concrete medians and to warn them about getting too close to the shoulder. In more recent focus group discussions conducted to identify intersection geometric design features that pose difficulty for older drivers and pedestrians (Staplin, Harkey, Lococo, and Tarawneh, 1997), drivers mentioned that they have problems seeing concrete barriers in the rain and at night, and characterized barriers as "an obstruction waiting to be hit."

G. Design Element: Curb Radius

Table 10. Cross-references of related entries for curb radius.

Applications in Standard Reference Manuals	
AASHTO Green Book (1994)	NCHRP 279 Intersection Channelization Design Guide (1985)
Pgs. 665-672, Sect. on *Effect of Curb Radii on Turning Paths*	Pg. 20, Bottom Fig. Pg. 21, Fig. 3-1 Pg. 26, Bottom Fig. Pg. 36, Middle Fig. Pgs. 66-69, Sect. on *Corner Radius Design* Pg. 73, Fig. 4-29

Curb radii, simply defined, are the radii of curves that join the curbs of adjacent approaches. The size of the radii affects the size of vehicle that can turn at the intersection, the speed at which vehicles can turn, and the width of intersection that must be crossed by pedestrians. If the curb radii are too small, lane encroachments resulting in traffic conflicts and increased accident potential can occur. If the radii are too large, pedestrian exposure may be increased (although, if large enough, refuge islands may be provided). The procedures used in the design of curb radii are well detailed in the Green Book (AASHTO, 1994).

McKnight and Stewart (1990) identified the task of positioning a vehicle in preparation for turning as a critical competency. A significant problem identified in a task analysis to prioritize older drivers' problems with intersections is carrying out the tight, right-turn maneuver at normal travel speed on a green light (Staplin, Lococo, McKnight, McKnight, and Odenheimer, 1994). The problems are somewhat moderated when right turns are initiated from a stop, because the turn can be made more slowly, which reduces difficulties with short radii. Older drivers may seek to increase the turning radius by moving to the left before initiating the turn, often miscommunicating an intent to turn left and encouraging following drivers to pass on the right. Or, they may initiate the turn from the correct position, but swing wide into a far lane in completing the turn in order to lengthen the turning radius and thus minimize rotation of the steering wheel. Encroaching upon a far lane can lead to conflict with vehicles approaching from the right or, on multilane roads, oncoming drivers turning to their left at the same time. The third possibility is to cut across the apex of the turn, possibly dragging the rear wheels over the curb. Each of these shortcomings in lanekeeping can be overcome by a channelized right-turn lane or wider curb radii.

Chu (1994) found that relative to middle-aged drivers (ages 25–64), older drivers (age 65 and older) tend to drive larger automobiles and drive at slower speeds. Although large heavy cars are associated with an accident fatality rate that is less than one-quarter of that associated with the smallest passenger cars (Insurance Institute for Highway Safety, 1991) and are, therefore, a wise choice for older drivers who are more frail than their middle-aged counterparts, large vehicles have a larger turning radius, which may exacerbate the problems older drivers exhibit in lanekeeping during a turn. Roberts and Roberts (1993) reported that common arthritic illnesses such as osteoarthritis, which affects more than 50 percent of the elderly population, and rheumatoid arthritis, affecting 1 to 2 percent, are relevant to the tasks of turning and gripping the steering wheel. A hand deformity caused by either osteoarthritis or rheumatoid arthritis may be very sensitive to pressure, making the driver unwilling to apply full strength to the steering wheel or other controls. In an assessment of 83 drivers with arthritis, Cornwell (1987) found that 83 percent of the arthritic group used both hands to steer, 7 percent used the right hand only, and 10 percent the left hand only; in this study, more than one-half of the arthritic group required steering modifications, either in the form of power steering or other assistive device such as a smaller steering wheel.

The *Intersection Channelization Design Guide* (Neuman, 1985) states that intersections on high-speed roadways with smooth alignment should be designed with sufficient radii to accommodate moderate- to high-speed turns. At other intersections, such as in residential neighborhoods, low-speed turns are desirable, and smaller corner radii are appropriate in these cases. Additionally, selection of a design vehicle is generally based on the largest standard or typical vehicle type that would regularly use the intersection. For example, a corner radius of 15 m (50 ft) will accommodate moderate-speed turns for all vehicles up to WB-50 (combination truck/large semitrailer with an overall length of 17 m [55 ft]). However, many agencies are designing intersections along their primary systems to accommodate a 21-m (70-ft), single trailer design vehicle (C-70). Table 4-8 (p. 66) of the *Intersection Channelization Design Guide* provides guidelines for the selection of a design vehicle. It further specifies in table 4-9 what the operational characteristics are of various corner radii. For example, a corner radius of less than 1.5 m (5 ft) is not appropriate even for P design vehicles (passenger cars), whereas a corner radius of 6–9 m (20–30 ft) will accommodate a low-speed turn for P vehicles, and a crawl-speed turn for SU vehicles (single unit truck, 9 m [30 ft] in length) with minor lane encroachment.

Of equal importance to the consideration of the right-turning design vehicle in determining curb radii is a consideration of pedestrian crossing time, particularly in urban areas. Smaller corner radii (less than 9 m [30 ft]) can decrease right-turn speeds and reduce open pavement area for pedestrians crossing the street. A consideration of vehicle turning speed and pedestrian crossing distance can contribute to the safe handling of vehicle/pedestrian crossing conflicts (Neuman, 1985). Hauer (1988) noted that "the larger the curb-curve radius, the larger the distance the pedestrian has to cover when crossing the road. Thus, for a sidewalk whose centerline is 1.8 m (6 ft) from the roadway edge, a 4.5-m (15-ft) corner radius increases the crossing distance by only 1 m (3 ft). However, a 15-m (50-ft) radius increases this distance by 8 m (26 ft), or 7 s of additional walking time." Hauer further stated that the following are widely held concerns with the widening of curb radii: (1) the longer the crossing distance, the greater the hazard to pedestrians, even though there may be space for refuge islands when the corner radius is large enough; (2) larger curb radii may induce drivers to negotiate the right turn at a higher speed; and (3) the larger the radius, the wider the turn, which makes it more difficult for the driver and the pedestrian to see each other. For these reasons, the safety of older persons at intersections, particularly pedestrians, may be adversely affected when large curb radii are provided.

In focus group discussions with 46 drivers ages 65–74 (young-old group) and 35 drivers age 75 and older (old-old group), 74 percent of drivers in each age group reported that tight intersection corner radii posed difficulty in maneuvering through the intersection for the following reasons: (1) there are visibility problem with sharp corners; (2) drivers sometimes hit curbs and median barriers; and (3) with sharp turns, trucks turning left into the adjacent opposing traffic lane end up face-to-face with drivers, requiring them to back up (Staplin, Harkey, Lococo, and Tarawneh, 1997). Approximately 24 percent of the young-old drivers and 34 percent of the old-old drivers suggested that medium rounding is sufficient to facilitate turning maneuvers and is safer than very broadly rounded corners because the latter encourages high-speed turns.

In a design preference study using slides to depict varying radii of corner curb cuts, four alternative curb geometries were presented to 30 drivers ages 65–74 (young-old group) and 30 drivers age 75 and older (old-old group) (Staplin et al., 1997). The four alternative geometries (depicted in figure 5) were: (1) a simple circular radius of 5.5 m (18 ft); (2) a simple circular radius of 12 m (40 ft); (3) a simple circular radius of 14.6 m (48 ft); and (4) a three-sided/truncated curve with the center side measuring 16.5 m (54 ft).

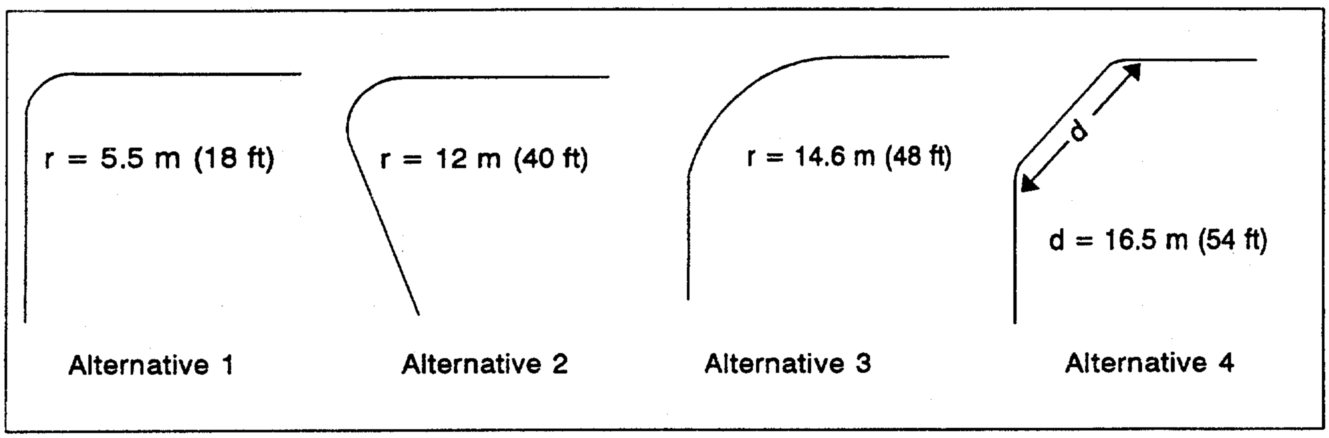


Figure 5. Alternative curb radii evaluated in laboratory preference study conducted by Staplin et al. (1997).

The alternatives were identically ranked by both older samples: Alternative 3 was consistently preferred, Alternative 4 placed second, Alternative 2 placed third, and Alternative 1 was least preferred. Both young-old and old-old drivers in this study were most concerned about ease of turning, citing the better maneuverability and less chance of hitting the curb as their primary basis of response. The second most common—but also strongly weighted—reason for the preference responses of both groups related to the degree of visibility of traffic on intersecting roadways, possibly explaining the slight preference for Alternative 2 over Alternative 1. Alternatives 3 and 4 both are described by corner curbline geometries offering ease of turning and good visibility; however, isolated responses to the truncated corner geometry (Alternative 4) indicated concerns that *too much* room in the right-turn path might result in a lack of needed guidance information and could lead to a maneuver error, and that it could be harder to detect pedestrians with this design.

In a field study conducted as part of the same project, three intersections providing right-turn curb radii of 12.2 m, 7.6 m, and 4.6 m (40 ft, 25 ft, and 15 ft) were evaluated to examine the effects of curb radii on the turning paths of vehicles driven by drivers in three age groups. One hundred subjects divided across three age groups drove their own vehicles around test routes using the local street network in Arlington, VA. The three age groups were "young/middle-aged" (ages 25-45), which contained 32 drivers; "young-old" (ages 65-74), containing 36 drivers; and "old-old" (age 75 and older), containing 32 drivers. The speed limit was 56 km/h (35 mi/h) and all intersections were located on major or minor arterials within a growing urban area. Data were only collected for turns executed on a green-signal phase.

Analysis of the free-flow speeds showed that all factors (age, gender, and geometry), and their interactions, were significant. Mean free-flow speeds were highest at the largest (12.2-m [40-ft]) curb radius location, for all age groups. A consistent finding showed that the slowest mean free-flow speeds were measured at the 4.6-m (15-ft) curb radius location for all age groups. Thus, larger curb radii increased the turning speeds of all drivers, with young/middle-aged and young-old drivers traveling faster than old-old drivers when making right turns.

H. Design Element: Traffic Control for Left-Turn Movements at Signalized Intersections

Table 11. Cross-references of related entries for left-turn movements at signalized intersections.

Applications in Standard Reference Manuals		
MUTCD (1988)	AASHTO Green Book (1994)	NCHRP 279 Intersection Channelization Design Guide (1985)
Pg. 4B-2, Para. 4, Items 1a & 1b Pg. 4B-3, Items 2a & 3b Pgs. 4B-4 - 4B-6, Items 4c under Sect. 4B-5 and 1b & 3-7 under Sect. 4B-6 Pg. 4B-6, Items 1 and 4 Pg. 4B-7, Para(s) 3-4 Pgs. 4B-8 & 4B-9, Sect. on *Arrangement of Lenses in Signal Faces* Pg. 4B-12, Entire page Pgs. 4B-15 & 4B-16, Sect. on *Vehicle Change Interval*	Pg. 319, Para. 2 Pg. 637, Para(s). 7-8 Pg. 640, Figure IX-7 Pg. 641, Para. 1 Pg. 847, Para. 1 Pgs. 852-860, Sect. on *Single-Point Diamond*	Pg. 1, Item 3, 4th Bullet Pg. 21, Fig. 3-1 Pg. 28, Top Fig. Pg. 29, Top Left Fig. Pg. 33, Bottom Left Fig. Pg. 36, Top & Bottom Fig(s). Pg. 37, Para. 2 & Top Left Fig. Pgs. 47-52, Section on *Warrants and Guidelines for Use of Left-Turn Lanes* & Fig. 4-11 Pg. 61, Sect. on *Other Left-Turn Treatments* Pg. 62, Fig. 4-22

Accident analyses have shown that older drivers, ages 56–75 and age 76 and older, are overinvolved in left-turn maneuvers at signalized intersections, with failure to yield right-of-way or disregarding the signal the principal violation types (Staplin and Lyles, 1991; Council and Zegeer, 1992). Old-elderly drivers (age 75 and older) were more likely than younger drivers (ages 30–50) to be involved in left-turn accidents at urban signalized intersections, and both young-elderly (ages 65–74) and old-elderly were more likely to be involved in left-turn accidents at rural signalized intersections. In both cases, the accident-involved older drivers were more likely to be performing a left-turn maneuver than the younger drivers. In addition, Stamatiadis, Taylor, and McKelvey (1991) found that the relative accident involvement ratios for older drivers were higher at two-phase (no turning phase) signalized intersections than for multiphase (includes turn arrow) signalized intersections. This highlights problems older drivers may have determining acceptable gaps and maneuvering through traffic streams when there is no protective phase. Further, accident percentages increased significantly for older drivers when an intersection contained flashing controls, as opposed to conventional (red, yellow, green) operations. In this analysis, the greatest accident frequency at signalized intersections occurred on major streets with five lanes, followed closely by roadways containing four lanes. These configurations were most often associated with low-speed, high-volume urban locations, where intersection negotiation requires more complex decisions involving more conflict vehicles and more visually distracting conditions. Not surprisingly, Garber and Srinivasan's (1991) analysis of 7,000 intersection accidents involving drivers ages 50–64 and age 65 and older, found that the provision of a protected left-turn phase will aid in reducing the accident rates of the elderly at signalized intersections.

The change in the angular size of a moving object, such as an approaching vehicle observed by a driver about to turn left at an intersection, provides information crucial to gap judgments (i.e., speed and distance). Age-related declines (possibly exponential) in the ability to detect angular movement have been reported. Older persons may in fact require twice the rate of movement than younger persons to perceive that an object is approaching, given a brief (2.0 s) duration of exposure. Also, older persons participating in laboratory studies have been observed to require significantly longer than younger persons to perceive that a vehicle was moving closer at constant speed: at 31 km/h (19 mi/h), decision times increased 0.5 s between ages 20 and 75 (Hills, 1975)..

Compounding this age-related decline in motion perception, some research has indicated that, relative to younger subjects, older subjects underestimate approaching vehicle speeds (Hills and Johnson, 1980). Specifically, Scialfa, Guzy, Leibowitz, Garvey, and Tyrrell (1991) showed that older adults tend to overestimate approaching vehicle velocities at lower speeds and underestimate at higher speeds, relative to younger adults. Staplin, Lococo, and Sim (1993), while investigating causes of older driver overinvolvement in turning accidents at intersections, did not find evidence of overestimation of time-to-collision by older drivers in their perception of the closing distance between themselves and another vehicle approaching either head-on or on an intersecting path. However, a relative insensitivity to approach (conflict) vehicle speed was shown for older versus younger drivers, in that younger drivers adjusted their gap judgment of the "last safe moment" to proceed with a turn appropriately to take higher approach speeds into account, while older drivers as a group failed to allow a larger gap for a vehicle approaching at 96 km/h (60 mi/h) than for one approaching at 48 km/h (30 mi/h). The interpretation of this and other data in this study was that older drivers rely primarily or exclusively on perceived vehicle separation distance to reach maneuver "go/no go" decisions, reflecting a reduced ability to integrate time and distance information with increasing age. Thus, a principal source of risk at intersections is the error of an older, turning driver in judging gaps in front of fast vehicles.

Aside from (conflict vehicle) motion detection, an additional concern is whether there are age differences in how well drivers understand the rules under which the turns will be made—that is, whether older drivers have disproportionately greater difficulty in understanding the message that is being conveyed by the signal and any ancillary (advisory) signs. If the signals and markings are not understood, at a minimum there may be delay in making a turn or, in the worst case, an accident could result if a protected operation is assumed where it does not exist.

A driver comprehension analysis conducted in a laboratory setting with drivers 30–60 years of age and older showed that green displays (those with the green ball alone, green arrow alone, or combinations of green ball and green arrow on the left-turn signal) were correctly interpreted with widely varying frequency, depending on the signals shown for the turning and through movements (Curtis, Opiela, and Guell, 1988). In most cases, performance declined as age increased; older drivers were correct approximately half as often as the youngest drivers. Most driver errors, and especially older driver errors, indicated signal display interpretations that would result in conservative behavior, such as stopping and/or waiting. A summary of the results follows. The simple green ball under permitted control was correctly interpreted by approximately 60 percent of the subjects. For protected-only operations, the green arrow (with red ball for through movement) was correctly answered by approximately 75 percent of drivers. For protected/permitted operation, the green ball alone was correctly answered by only 50 percent of the respondents, while the green arrow in combination with the green ball had approximately 70 percent correct responses. When the green ball with the green arrow was supplemented by the R10-12 sign LEFT TURN YIELD ON GREEN ●, only 34 percent of drivers answered correctly. This test result suggests that the MUTCD recommended practice may result in some driver confusion, as test subjects answered correctly more often when the sign was *not* present, even when the effects of regional differences in familiarity with the sign were considered. Green arrows were better understood than green balls. Conversely, red and yellow arrows were less comprehensible than red and yellow balls. Potentially unsafe interpretations were found for red arrow displays in protected-only operations. The yellow arrow display was more often treated as a last chance to complete a turn when compared with a yellow ball. Driver errors were most frequent in displays that involved flashing operations, and multiple faces with different colors illuminated on the left-turn signal head, and in particular, different colors on the turn and through signals.

When Hummer, Montgomery, and Sinha (1990) evaluated motorists' understanding of left-turn signal alternatives, they found that the protected-only signal was by far the best understood, permitted signals were less understood, and the protected/permitted the least understood. When a green ball for through traffic and a green arrow for left turns were displayed, the protected signal was clearly preferred over the permitted and protected/permitted signals, and the leading signal sequence was preferred more often than the lagging sequence. Respondents stated that the protected-only signal caused less confusion, was safer, and caused less delay than the permitted and protected/permitted signals. It should be noted, however, that while older persons were in the sample of drivers studied, they made up a very small percentage (8 of 402) and differences were hard to substantiate.

More recently, Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin (1995) examined the lack of understanding associated with a variety of protected and permitted left-turn signal displays. They found that many drivers, both younger and older, do not understand the protected/permitted signal phasing, and they suggested that efforts to improve motorist comprehension of left-turn signal phasing should be targeted at the entire driving population. In focus group discussions, many older drivers reported that they avoid intersections that do not have a protected-only (left-turn arrow) phase or those where the time allowance for left turns was too short. In addition, the situation where the green arrow eventually turns to a solid green ball was generally confusing and not appreciated by the older participants. Among the recommendations made by the older drivers were:

- Provide as many protected left-turn opportunities as possible.
- Standardize the sequence for the left-turn green arrow so that it *precedes* solid green or red.
- Lengthen the protected left-turn signal.
- Lengthen the left-turn storage lanes so that turning traffic does not block through traffic.
- Make traffic signal displays more uniform across the United States, including the warning or amber phase.
- Standardize the position and size of signals.
- Provide traffic lights overhead and to the side at major intersections.

- Paint a yellow line in the pavement upstream of the signal in a manner that, if the driver has not reached the line before the light has turned yellow, he/she cannot make it through before the red light.
- Provide borders around lights to minimize glare from the sun.
- Eliminate decorations on signal heads, because they are often green and red and may be confusing near signal faces.

Bonneson and McCoy (1994) also found a decreased understanding of protected and permitted left-turn designs with increased age, in a survey conducted in Nebraska with 1,610 drivers. In this study, the overlap phase (left-turn green arrow and through green ball illuminated) was the least understood by drivers wishing to turn left, with only one-half of the respondents answering correctly; most of the respondents who erred chose the safer course of action, which was to wait for a gap in oncoming traffic. In terms of signal head location, 4 to 5 percent more drivers were able to understand the protected/permitted display when it was centered in the left-turn lane (exclusive) as opposed to having the head located over the lane line (shared). Although the difference was statistically significant, Bonneson and McCoy point out that the difference may be too small to be of practical significance. In terms of lens arrangement, significantly more drivers understood both the permitted indication and the protected/MUTCD indication (left-turn green arrow and through red ball) in vertical and horizontal arrangements than in the cluster arrangement. An analysis of sign use compared the exclusive cluster lens arrangement over the left-turn lane and exclusive vertical lens arrangement over the through lanes with and without the use of an auxiliary sign (LEFT TURN YIELD ON GREEN ●). Overall, the results indicated that driver understanding of the display increased by about 6 percent when there was *no* sign, though a closer examination of these data revealed that the specific operation signaled by the display was critical. For the permitted indication, the sign appeared to help driver understanding, whereas during the overlap and protected indications it appeared to confuse drivers. Comparisons between the protected/MUTCD indication and a modified protected indication (green arrow with no red ball), showed that for the horizontal protected/permitted designs, 25 percent more drivers were able to understand the protected indication when the red ball was *not* shown with the green arrow, and for the vertical and cluster protected/permitted designs, 12 percent more drivers understood the modified protected indication. The point is that from an operational perspective, hesitancy as a result of misunderstanding will decrease the level of service and possibly result in accident situations.

Numerous studies have found that: (1) protected left-turn control is the safest, with protected/permitted being less safe than protected, but safer than permitted (Fambro and Woods, 1981; Matthais and Upchurch, 1985; Curtis et al., 1988); and (2) transitions from protected-only operations to protected/permitted operations experience accident increases (Cottrell and Allen, 1982; Florida Section of Institute of Transportation Engineers, 1982; Cottrell, 1985; Warren, 1985; Agent, 1987). According to Fambro and Woods (1981), for every left-turn accident during a protected phase, 10 would have occurred without protection. Before-and-after studies where intersections were changed from protected to permitted control have shown four- to sevenfold increases in left-turn accidents (Florida Section of Institute of Transportation Engineers, 1982; Agent, 1987).

Williams, Ardekani, and Asante (1992) conducted a mail survey of 894 drivers in Texas to assess motorists' understanding of left-turn signal indications and accompanying auxiliary signs. Drivers older than age 65 had the highest percentage of incorrect responses (35 percent). Results of the various analyses are as follows: (1) the use of a green arrow for protected-only left turns produced better comprehension than the use of a circular green indication, even when the circular green indication was accompanied by an auxiliary sign; (2) for a five-section signal head configuration, the display of a green left-turn arrow in isolation produced better driver understanding than the simultaneous display of a circular red indication and a green left-turn arrow; (3) the LEFT TURN YIELD ON GREEN ● auxiliary sign was associated with the smallest percentage of incorrect responses, compared with the LEFT TURN ON GREEN AFTER YIELD sign, the PROTECTED LEFT ON GREEN sign, and the LEFT TURN SIGNAL sign; and (4) the percentage of incorrect responses was 50 percent lower in the presence of a circular red indication compared with a red arrow; the red arrow was often perceived to indicate that a driver may proceed with caution to make a permitted left turn.

In another study conducted by Curtis et al. (1988), it was found that the Delaware flashing red arrow was not correctly answered by any subject. The incorrect responses indicated conservative interpretations of the signal displays which would probably be associated with delay and may also be related to rear-end collisions. Drivers interpreted the Delaware signal as requiring a full stop before turning, because a red indication usually means "stop," even though the signal is meant to remind motorists to exercise caution but not necessarily to stop unless opposing through traffic is present. Hulbert, Beers, and Fowler (1979) found a significant difference in the percentage of drivers younger than age 49 versus those older than age 49 who chose the correct meaning of the red arrow display. Sixty-one percent of the drivers older than age 49 chose "no turning left" compared with 76 percent of those younger than age 49. Although other research has concluded that the left-turn arrow is more effective than the red ball in some left-turn situations in particular jurisdictions where special turn signals and exclusive turn lanes are provided (Noel, Gerbig, and Lakew, 1982), drivers of all ages will be better served if signal indications are consistent. Therefore, it is recommended that the use of the arrow be reserved for protected turning movements and the color red be reserved for circular indications to mean "stop."

Hawkins, Womak, and Mounce (1993) surveyed 1,745 drivers in Texas to evaluate driver comprehension of selected traffic control devices. The sample contained 88 drivers age 65 and older. Three alternative signs describing the left-turn decision rule were evaluated: (1) R10-9, PROTECTED LEFT ON GREEN ARROW (in the Texas MUTCD but not the national MUTCD); (2) R10-9a, PROTECTED LEFT ON GREEN (in the Texas MUTCD but not the national MUTCD); and (3) R10-12, LEFT TURN YIELD ON GREEN ●. The R10-12 sign did the best job of the signs in the survey informing the driver of a permitted left-turn condition, with 74.5 percent choosing the desirable response. Of those who responded incorrectly, 13.6 responded that they would wait for the green arrow, and 4.3 percent made the dangerous interpretation that the left turn was protected when the green ball was illuminated. Incorrect responses were more often made by drivers age 65 and older.

The decisional processes drawing upon working memory crucial to safe performance at intersections may be illustrated through a study of alternative strategies for presentation of left-turn traffic control messages (Staplin and Fisk, 1991). This study evaluated the effect of providing advance left-turn information to drivers who must decide whether or not they have the right-of-way to proceed with a protected turn at an intersection. Younger (mean age of 37) and older (mean age of 71) drivers were tested using slide animation to simulate dynamic approaches to intersection traffic control displays, with and without advance cueing of the "decision rule" (e.g., LEFT TURN MUST YIELD ON GREEN ●) during the intersection approach. Without advance cueing, the decision rule was presented only on a sign mounted on the signal arm across the intersection as per standard practice, and thus was not legible until the driver actually reached the decision point for the turning maneuver. Cueing drivers with advance notice of the decision rule through a redundant upstream posting of sign elements significantly improved both the accuracy and latency of all drivers' decisions for a "go/no go" response upon reaching the intersection, and was of particular benefit to the older test subjects. Presumably, the benefit of upstream "priming" is derived from a reduction in the requirements for serial processing of concurrent information sources (sign message and signal condition) at the instant a maneuver decision must be completed and an action performed.

Stelmach, Goggin, and Garcia-Colera (1987) found that older adults were particularly impaired when preparation was not possible, showing disproportionate response slowing when compared with younger subjects. When subjects obtained full information about an upcoming response, reaction time (RT) was faster in all age groups. Stelmach et al. (1987) concluded that older drivers may be particularly disadvantaged when they are required to initiate a movement in which there is no opportunity to prepare a response. Preparatory intervals and length of precue viewing times are determining factors in age-related differences in movement preparation and planning (Goggin, Stelmach, and Amrhein, 1989). When preparatory intervals are manipulated in a way that older adults have longer stimulus exposure and longer intervals between stimuli, they profit from the longer inspection times by performing better and exhibiting less slowness of movement (Eisdorfer, 1975; Goggin et al., 1989). Since older drivers benefit from longer exposure to stimuli, Winter (1985) proposed that signs should be spaced farther apart to allow drivers enough time to view information and decide what action to take. Increased viewing time will reduce response uncertainty and decrease older drivers' RT.

The differences in maneuver decision responses demonstrated in the Staplin and Fisk (1991) study illustrate both the potential problems older drivers may experience at intersections due to working memory deficits, and the possibility that such consequences of normal aging can to some extent be ameliorated through improved engineering design practices. Staplin and Fisk (1991) also showed that older drivers had higher error rates and increased decision latencies for situations where the left turn was not protected. In particular, the most problematic displays were those with only one steady illuminated signal face (green ball) accompanied by a sign that indicated that it was *not* safe to proceed into the intersection with the assumption of right-of-way (LEFT TURN YIELD ON GREEN ●). A correct response to this combination depends on the inhibition of previously learned "automatic" responses; a signal element with one behavior (go) was incorporated into a traffic control display requiring another, conflicting behavior.

Hummer, Montgomery, and Sinha (1991) evaluated leading and lagging signal sequences using a survey of licensed drivers in Indiana, an examination of traffic conflicts, an analysis of accident records, and a simulation model of traffic flow, to evaluate motorists' understanding and preference for leading and lagging schemes as well as determining the safety and delay associated with each scheme. Combinations of permitted and protected schemes included:
(1) protected-only/leading, in which the protected signal is given to vehicles turning left from a particular street before the green ball is given to the through movement on the same street;

(2) protected-only/lagging, in which the green arrow is given to left-turning vehicles after the through movements have been serviced; (3) protected/permitted, in which protected left turns are made in the first cycle and a green-ball signal allows permitted turns later in the cycle; and (4) permitted/protected, in which permitted turns are allowed first in the cycle and protected left turns are accommodated later in the cycle. The protected-only/leading and protected/permitted schemes are known as "leading," and the protected-only/lagging and permitted/protected are known as "lagging" schemes. Of the 402 valid responses received, 248 respondents preferred the leading, 59 preferred the lagging sequence, and 95 expressed no preference. The most frequent reasons given for preference of the leading sequence were: it is more like normal; it results in less delay; and it is safer. There are apparent tradeoffs here, however; the leading sequence was associated with a higher conflict rate with pedestrians and a higher rate of run-the-red conflicts (drivers turning left during the clearance interval for opposing traffic), while the intersections with a lagging sequence were associated with a significantly higher rate of indecision conflicts than the leading intersections due to violations in driver expectancy. Overall, it is judged that consistency in signal phasing across intersections within a jurisdiction, as well as across jurisdictions, should be a priority, and that use of a leading protected left-turn phase offers the most benefits. A discussion of countermeasures for the protection of pedestrians may be found in the material that presents the Rationale and Supporting Evidence for Design Elements I and P.

Upchurch (1991) compared the relative safety of 5 types of left-turn phasing using Arizona Department of Transportation accident statistics for 523 intersection approaches, where all approaches had a separate left-turn lane, 329 approaches had 2 opposing lanes of traffic, and 194 approaches had 3 opposing lanes. The five types of left-turn phasing included (1) permitted, (2) leading protected/permitted, (3) lagging protected/permitted, (4) leading protected-only, and (5) lagging protected-only. For the 495 signalized intersections in the State highway system, most samples represented a 4-year accident history (1983–1986). For the 132 signalized intersections in 6 local jurisdictions in Arizona, samples ranged from 4 months to 4 years, all between 1981 and 1989. When the accident statistics were stratified by various ranges of left-turn volume and various ranges of opposing volume (vehicles per day), the following observations and conclusions were made for sample sizes greater than five, eliminating any conclusions about lagging protected-only phasing:

- Leading protected-only phasing had the lowest left-turn accident rate in almost every case. This was true in every left-turn volume range and every opposing volume range except one (19 out of 20 cases). Lagging protected/permitted was the exception for 3 opposing lanes and left-turn volumes of 0–1,000.
- When there were two lanes of opposing traffic, lagging protected/permitted tended to have the worst accident rate.
- When there were three lanes of opposing traffic, leading protected/permitted tended to have the worst accident rate.
- When there were two lanes of opposing traffic, the order of safety (accident rate from best to worst) was leading protected-only, permitted, leading protected/permitted, and lagging protected/permitted. However, there was a small difference in the accident rate among the last three types of phasing.

- When there were three lanes of opposing traffic, the order of safety (accident rate from best to worst) was leading protected-only, lagging protected/permitted, permitted, and leading protected/permitted.

Upchurch (1991) compared the accident experience of 194 intersections that had been converted from one type of phasing to another in a simple before-and-after design. For each conversion, 4 years of before-accident data and 4 years of after-accident data were used, where available. At approaches having *two* opposing lanes of traffic, the statistics for conversions from permitted to leading protected/permitted and vice versa reinforced each other, suggesting that leading protected/permitted is safer than permitted. At approaches having *three* opposing lanes of traffic, the statistics for conversions from leading protected-only to leading protected/permitted and vice versa reinforced each other, suggesting that leading protected-only is safer than leading protected/permitted.

Parsonson (1992) stated that a lagging left-turn phase should be used only if the bay provides sufficient storage, as any overflow of the bay during the preceding through-movement will spill into the adjacent through lane, blocking it. A lag should also be reserved for those situations in which opposing left-turn movements (or U-turns) are safe from the left-turn trap (or are prohibited). The "left-turn trap" occurs when the left-turning driver's right-of-way is terminated, while the opposing (oncoming) approach continues with a green arrow and an adjacent through movement. Thus, left-turning drivers facing a yellow indication are trapped; they believe that the opposing traffic will also have a yellow signal, allowing them to turn on the yellow or immediately after. Since the opposing traffic is not stopping, the turning driver is faced with a potentially hazardous situation. Locations where the left-turn trap is not a hazard include T-intersections, and those where the left turn (or U-turn) opposing the green arrow is prohibited or is allowed only on a green arrow (protected-only phasing). In addition, driver expectancy weighs heavily in favor of leading left turns, and driver confusion over lagging left turns results in losses in start-up time.

I. Design Element: Traffic Control for Right-Turn/RTOR Movements at Signalized Intersections

Table 12. Cross-references of related entries for right turn/RTOR movements at signalized intersections.

Applications in Standard Reference Manuals		
MUTCD (1988)	AASHTO Green Book (1994)	NCHRP 279 Intersection Channelization Design Guide (1985)
Pg. 4B-2, Para. 4, Items 1a & 1b Pg. 4B-3, Items 2a, 3b & 3c Pgs. 4B-4 - 4B-6, Items 4c under Sect. 4B-5 and 1b & 4-7 under Sect. 4-6 Pg. 4B-6, Items 1 and 4 Pg. 4B-7, Para(s) 3-4 Pgs. 4B-8 & 4B-9, Sect. on *Arrangement of Lenses in Signal Faces* Pg. 4B-13, Item 4 Pgs. 4B-15 & 4B-16, Sect. on *Vehicle Change Interval*	Pg. 211, Para. 3 Pg. 319, Para. 2 Pg. 637, Para. 8 Pg. 718, Para. 2 Pg. 719, Para. 3	Pg. 28, Middle Fig. Pg. 33, Bottom Left Fig. Pg. 29, Top Right Fig. Pg. 36, Top Fig. Pgs. 38-39 Pgs. 61-65, Sect. on *Exclusive Right-Turn Lanes*

The right-turn-on-red (RTOR) maneuver provides increased capacity and operational efficiency at a low cost (Institute of Transportation Engineers [ITE], 1992). However, traffic control device violations and limited sight distances need to be addressed in order to reduce the potential for safety problems. ITE concluded that a significant proportion of drivers do not make a complete stop before executing an RTOR, and a significant portion of drivers do not yield to pedestrians. In a review of the literature on RTOR laws and motor vehicle crashes, Zador (1984) reported findings that linked RTOR to a 23 percent increase in all right-turning crashes, a 60 percent increase in pedestrian crashes, and a 100 percent increase in bicyclist crashes. Analysis of police accident reports in four States indicated that drivers who are stopped at a red light are looking left for a gap in traffic and do not see pedestrians and bicyclists coming from their right (Preusser, Leaf, DeBartolo, and Levy, 1982). Eldritch (1989) noted that, adding to the adverse effects RTOR has on pedestrian accidents, many motorists persist in making right turns on red even when there is a sign that prohibits the maneuver.

The most recent data available on the safety impact of RTOR were provided by Compton and Milton (1994) in a report to Congress by the National Highway Traffic Safety Administration. Using Fatal Accident Reporting System (FARS) data and data from four State files for 1989–1992, it was concluded that RTOR crashes represented a small proportion of the total number of traffic crashes in the four States (0.05 percent) and of all fatal (0.03 percent), injury (0.06 percent), and signalized-intersection crashes (0.40 percent). FARS data showed that approximately 84 fatal crashes per year occurred involving a right-turning vehicle at an intersection where RTOR is permitted; however, because the status of the traffic signal indication is not available in this database, the actual number of fatal crashes that occurred when the signal was red is not known. Slightly less than one-half of these crashes involved a pedestrian (44 percent), 10 percent involved a bicyclist, and 33 percent involved one vehicle striking another. Although no data on the age of the drivers involved in RTOR crashes were provided, there are reasons for concern that increasing problems with this maneuver may be observed with the dramatic growth in the number of older drivers in the United States.

The difficulties that older drivers are likely to experience making right turns at intersections are a function of their diminishing gap-judgment abilities, reduced neck/trunk flexibility, attention-sharing deficits, slower acceleration profile, and their general reduction in understanding traffic control devices compared with younger drivers. Right-turning drivers face possible conflicts with pedestrians, and restrictions in the visual attention of older drivers may allow pedestrian and vehicular traffic to go unnoticed. The fact that pedestrians may be crossing the side street, where they enter the path of the right-turning vehicle, places a burden upon the driver to search the right-turning path ahead. The result is the need to share attention between oncoming vehicles approaching from the left and pedestrians in the path to the right. Limitations in the range of visual attention, frequently referred to as "useful field of vision," further contribute to the difficulty of older drivers in detecting the presence of pedestrians or other vehicles near the driver's path. Older drivers, who may have greater difficulty maintaining rapid eye movements and associated head movements, are less likely to make correct judgments on the presence of pedestrians in a crosswalk or on their walking speed (Habib, 1980).

Researchers have identified that the right-turn maneuver is more problematic for older drivers compared with young or middle-aged drivers, presumably as a result of age-related diminished visual, cognitive, and physical capabilities. Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin (1995) conducted an analysis of right-angle, left-turning, right-turning, sideswipe, and rear-end accidents at intersections in Minnesota and Illinois for the time period of 1985–1987, comparing accident proportions and characteristics of "middle-aged" drivers ages 30–50, "young-elderly" drivers ages 65–74, and "old-elderly" drivers age 75 and older. Turning right accounted for 35.8, 39.3, and 42.9 percent, respectively, of the middle-aged, young-elderly, and old-elderly drivers' accidents at urban locations. It appears that, for right-turning accidents, the middle-aged driver is most likely crossing the intersection on a green signal and the older drivers are turning right on a red signal in front of the oncoming middle-aged driver. Similar patterns emerged from examination of the rural signalized-intersection precrash maneuvers, with middle-aged drivers most often traveling straight, and older drivers most often turning left or right. Looking at the contributing factors in angle and turning collisions for both rural and urban signalized locations, the middle-aged group was much more likely to be characterized by the police officer as having exhibited "no improper driving." This occurred in 65 percent of the accidents involving this age group, compared with 30.7 percent of the young-elderly, and 13.4 percent of the old-elderly. The two elderly groups were more likely to be cited for failing to yield (42.0 percent of the old-elderly, 31.9 percent of the young-elderly, and 10.9 percent of the middle-aged); disregarding the traffic control device (30.7 percent of the old-elderly, 22.0 percent of the young-elderly, and 10.3 percent of the middle-aged); and driver inattention (8.2 percent of the old-elderly, 8.9 percent of the young-elderly, and 6.4 percent of the middle-aged).

Knowledge testing has indicated that, compared with younger drivers, older drivers are less familiar with the meaning of traffic control devices and relatively new traffic laws (McKnight, Simone, and Weidman, 1982). "Newness" of traffic laws, in this regard, relates not to the period of time that has elapsed since the device or law was implemented, but the low frequency with which drivers come in contact with the situation. Older drivers may not encounter right turn on red after stop (RTOR), no turn on red (NTOR), or red right-turn arrow situations on a daily basis, due to the significantly lower amount and frequency of driving in which they are engaged.

Hulbert, Beers, and Fowler (1979) found that when presented with a red arrow pointing right, only 75 percent of drivers across all age groups chose the correct answer (no right turn on red). There was a significant difference in the number of correct responses between drivers age 50 and older and drivers younger than age 50. The younger drivers gave the correct answer 80 percent of the time, whereas the older drivers were correct 66 percent of the time. Twenty-four percent of the older drivers thought they were permitted to turn right after coming to a stop, as did 14 percent of the drivers younger than age 50. Owolabi and Noel (1985) also determined that the right-turn red arrow is not a safe traffic control device. In their study, the red ball received significantly fewer violations than the arrow when used for right turns, regardless of the time of day.

Knoblauch et al. (1995) found that both drivers younger than the age of 65 and drivers age 65 and older failed to understand that they could turn right on a red ball after stopping in the right lane. Although the survey indicated that older drivers were more likely to stop and remain stopped (45 percent) than younger drivers (36 percent), the differences were not significant.

Staplin, Harkey, Lococo, and Tarawneh (1997) conducted a controlled field study to measure differences in drivers' RTOR behavior as a function of driver age and right-turn lane channelization. In this study, 100 subjects divided across 3 age groups were observed as they drove their own vehicles around test routes using the local street network in Arlington, Virginia. The three age groups were young/middle-aged (ages 25–45), young-old (ages 65–74), and old-old (age 75+). The percentage of drivers who made RTOR at the four intersections was included as a measure of mobility.

Staplin et al. (1997) found that significantly fewer drivers in the old-old driver group attempted to make an RTOR (16 percent), compared with young/middle-aged drivers (83 percent) and young-old drivers (45 percent). Similarly, young/middle-aged drivers made an RTOR nearly 80 percent of the time when they had the chance to do so, compared with nearly 36 percent for the young-old drivers and 15 percent for the old-old drivers. Drivers made significantly fewer RTOR's at the skewed channelized intersection than at the other three locations. Analysis of the percentage of drivers who made an RTOR without a complete stop showed that age, right-turn lane geometry, gender, and the age-by-geometry interaction were significant. Young/middle-aged drivers made an RTOR without a complete stop nearly 35 percent of the time, compared with nearly 25 percent for the young-old and 3 percent for the old-old drivers. Channelized intersections with or without exclusive acceleration lanes encouraged making an RTOR without a complete stop. The nonchannelized and the skewed locations showed the lowest percentage of RTOR's without a complete stop, and were not significantly different from each other. The three age groups showed significantly different performance. Old-old drivers almost always stopped before making an RTOR regardless of the right-turn lane geometry. In only 1 of 26 turns did an older driver *not* stop before making an RTOR; this occurred at the channelized right-turn lane with an acceleration lane. At the nonchannelized intersection (which was controlled by a STOP sign), 22 percent of the young/middle-aged drivers, 5 percent of the young-old drivers, and none of the old-old drivers performed an RTOR without a stop. Where an acceleration lane was available, 65 percent of the young/middle-aged drivers continued through without a complete stop, compared with 55 percent of the young-old drivers and 11 percent of the old-old drivers. The increased mobility exhibited by the younger drivers at the channelized right-turn lane locations (controlled by YIELD signs) was not exhibited by old-old drivers, who stopped in 19 of the 20 turns executed at the channelized locations. In summary, with increases in driver age, the likelihood of RTOR decreases to a very low level for the present cohort of old-old drivers, but when these individuals do engage in this behavior, they are virtually certain to come to a complete stop before initiating the maneuver. Therefore, the emphasis is to ensure adequate sight distance for the older turning driver, to provide sign and signal indications that are most easily understood by this group, and to prompt these motorists to devote adequate attention to pedestrians who may be in conflict with their turning maneuver.

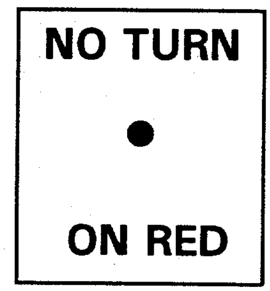


Figure 6. Novel sign tested as a countermeasure by Zegeer and Cynecki, 1986.

Zegeer and Cynecki (1986) found that offsetting the stop bar—moving the stop bar of adjacent stopped vehicles back from the intersection by 1.8 to 3.0 m (6 to 10 ft)—was effective in providing better sight distance to the left for RTOR motorists. It also reduced the RTOR conflicts with other traffic and resulted in more RTOR vehicles making a full stop behind the stop bar. The offset stop bar was recommended as a countermeasure for consideration at RTOR-allowed sites that have two or more lanes on an approach and heavy truck or bus traffic, or unusual geometrics. It was also found that a novel sign (red ball with NO TURN ON RED, shown in figure 6) was more effective than the standard black-and-white NO TURN ON RED (R10-11a) sign, and should be added to the MUTCD. They offered that the red ball on the sign helps draw drivers' attention to it, particularly as intersections are associated with a preponderance of signs and information. Increasing the size of the standard NO TURN ON RED sign from its present size of 600 mm x 750 mm (24 in x 30 in) to 750 mm x 900 mm (30 in x 36 in) reduced the proportion of violations at most of the test sites. Finally, Zegeer and Cynecki (1986) found that an electronic NO TURN ON RED blank-out sign was found to be slightly better than the standard MUTCD sign in terms of reducing violations, and it was effective in increasing RTOR maneuvers when RTOR was appropriate, thereby reducing vehicle delay. Although the sign is more expensive than standard signs and pavement markings, the authors concluded it may be justified in situations where pedestrian protection is critical during certain periods (i.e., school zones) or during a portion of the signal cycle when a separate, opposing left-turn phase may conflict with an unsuspecting RTOR motorist.

Several studies have been conducted to determine whether regulatory signing aimed at turning motorists could reduce conflicts with pedestrians. Zegeer, Opiela, and Cynecki (1983) found that the regulatory sign YIELD TO PEDESTRIANS WHEN TURNING was effective in reducing conflicts between turning vehicles and pedestrians. They recommended that this sign be added to the MUTCD as an option for use at locations with a high number of pedestrian accidents involving turning vehicles. Zegeer and Cynecki (1986) found that the standard NO TURN ON RED sign with the supplementary WHEN PEDESTRIANS ARE PRESENT message was effective at several sites with low to moderate right-turn vehicle volumes. However, it was less effective when RTOR volumes were high. It was therefore recommended that the supplemental message WHEN PEDESTRIANS ARE PRESENT be added to the MUTCD as an accepted message that may be used with an NTOR sign when right-turn volume is light to moderate and pedestrian volumes are light or occur primarily during intermittent periods, such as in school zones. The supplemental message when added to the NTOR red ball sign reduced total pedestrian conflicts at one site and increased RTOR usage (as desired, from 5.7 percent to 17.4 percent), compared with full-RTOR prohibitions. It was recommended that the supplemental message be added to the MUTCD for the NTOR red ball sign, under low to moderate right-turn vehicle volumes and light or intermittent pedestrian volumes.

More recently, Abdulsattar, Tarawneh, and McCoy (1996) found that the TURNING TRAFFIC MUST YIELD TO PEDESTRIANS sign was effective in significantly reducing pedestrian-vehicle conflicts during right turns. The sign was installed at six marked crosswalks in Nebraska, where right-turn vehicle-pedestrian conflict data were collected before and after its installation in an observational field study. For the six study crosswalks combined, a conflict occurred in 51 percent of the observations in the before period, but in only 38 percent of the observations during the after period. The reductions in pedestrian-vehicle conflicts across the observation sites ranged from 15 to 30 percent, and were statistically significant.

J. Design Element: Street-Name Signage

Table 13. Cross-references of related entries for street-name signage.

Applications in Standard Reference Manuals	
MUTCD (1988)	AASHTO Green Book (1994)
Pgs. 2D-1 - 2D-3, Sect(s). 2D-1 - 2D-6 Pgs. 2D-23 - 2D-24, Sect. 2D-39 Pg. 2E-9, Sect. on *Signs for Intersections at Grade*	Pg. 45, Para. 1 Pg. 314, Para(s). 2-3

The MUTCD (1988) states that the lettering on street-name signs (D3) should be at least 100 mm (4 in) high. Burnham (1992) noted that the selection of letter size for any sign must evaluate the needs of the user, which are continuously changing as a function of changes in automotive technology, the roadway system, and the population itself. It is estimated that by the year 2020, 17 percent or more of the population—nearly one in five—will be older than 65 years of age (Transportation Research Board, 1988). The ability to read street signs is dependent on visual acuity as well as divided attention capabilities, both of which decline significantly with advancing age.

Older drivers participating in focus groups and completing questionnaires for traffic safety researchers over the past decade have consistently stated that larger street signs with bigger lettering and standardization of sign placement overhead would make driving an easier task (Yee, 1985; Gutman and Milstein, 1988; Cooper, 1990; Staplin, Lococo, and Sim, 1990; Benekohal, Resende, Shim, Michaels, and Weeks, 1992; Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin, 1995). Problems with placement included signs that were either obstructed by trees, telephone poles, billboards, or large trucks, or placed too close to or across the intersection rather than on the near side. Older drivers stated that they needed more advance notice regarding upcoming cross streets and larger street-name signs placed overhead, to give them more time to make decisions about where to turn. Also noted were difficulties reading traffic signs with too much information in too small an area, and/or with too small a typeface, which results in the need to slow down or stop to read and respond to the sign's message. May (1992) noted that providing sufficient time to allow motorists to make appropriate turning movements when approaching cross streets can improve safety and reduce congestion, and that consistent street signing across political jurisdictions can be helpful in this regard, as well as presenting an orderly, predictable picture of the region to tourists, businesspeople, and residents.

Taoka (1991) discussed "spare glance" duration in terms of how drivers allocate their visual search time among different tasks/stimuli. The tasks ranged from side/rearview mirror glances during turning to reading roadway name signs. Although specific results were not differentiated by age, Taoka asserted that 85th percentile glance times at signs (about 2.4 s) were likely too long, as 2.0 s is the maximum that a driver should divert from the basic driving task. Since older drivers are more apt to be those drivers taking longest to read signs, these results imply that they will commonly have problems dividing attention between searching for/reading signs and the basic driving task. Malfetti and Winter (1987) observed that older drivers exhibited excessive vehicle-braking behavior whenever a signal or road sign was sighted. This was categorized as an unsafe behavior, because it is confusing and disruptive to following traffic when the lead vehicle brakes for no apparent reason. These researchers obtained many descriptions of older drivers who stopped suddenly at unexpected times and in unexpected places, frequently either within the intersection or 12 m (40 ft) before the intersection to read street signs.

The visibility of retroreflective signs must be considered with regard to their dual requirements of detection and legibility. The sign components affecting detection are sign size, color, shape, and message or content design. External factors affecting sign detection include its placement (e.g., left, right, or overhead), the visual complexity of the area, and the contrast of the sign with its background. The component parts of retroreflective signs that determine legibility fall into two major classes of variables: character and message. Character variables include the variables related to brightness—i.e., contrast, luminance, color, and contrast orientation—as well as font, letter height, letter width, case, and stroke width. Message variables address the visibility issues of spacing and include interletter, interword, interline, and copy-to-border distances.

Most studies of sign legibility report legibility distance and the letter height of the stimulus; dividing the former measure by the latter defines the "legibility index" (LI), which can serve as a common denominator upon which to compare different studies. Forbes and Holmes (1939) used the LI to describe the relative legibility of different letter styles. Under daytime conditions, series B, C, and D were reported to have indexes of 0.4 m/mm, 0.5 m/mm, and 0.6 m/mm (33, 42.5, and 50 ft/in), respectively. Forbes, Moskowitz, and Morgan (1950) found the wider, series E letters to have an index of 0.66 m/mm (55 ft/in). Over time the value of 0.6 m/mm (50 ft/in) of letter height has become the nominal, though arbitrary and disputed, standard. Based on the physical attributes of the older driver population, the current standard of 50 ft of legibility for every 1 in of letter height (corresponding to a visual acuity of 20/25) exceeds the visual ability of approximately 40 percent of the drivers between ages 65 and 74. The LI is important to the size requirement determination for a sign in a specific application.

Mace (1988), in his work on minimum required visibility distance (MRVD) for highway signs, noted the following relationships:

Required letter size = MRVD / LI *or* Required LI = MRVD / letter size

Either the letter size or the LI may be manipulated to satisfy the MRVD requirement, which specifies the minimum distance at which a sign should be read for proper driver reaction.

Olson and Bernstein (1979) suggested that older drivers should not be expected to achieve an LI of 0.6 m/mm (50 ft/in) under most nighttime circumstances. The data provided by this report gives some expectation that 0.48 m/mm (40 ft/in) is a reasonable goal under most conditions. A 0.48 m/mm (40 ft/in) standard can generally be effective for older drivers, given contrast ratios greater than 5:1 (slightly higher for guide signs) and luminance greater than 10 cd/m^2 for partially reflectorized signs. With regard to the effect of driver age on legibility, Olson, Sivak, and Egan (1983) concluded that older drivers require more contrast between the message and the sign's background than younger drivers to achieve the same level of comprehension. They also noted that legibility losses with age are greater at low levels of background luminance. A reduction in legibility distance of 10 to 20 percent should be assumed when signs are not fully reflectorized. Also, higher surround luminance improved the legibility of signs more for older drivers and reduced the negative effects of excessive contrast. In general, the LI for older drivers is 70 to 77 percent that of younger drivers. The average LI for older drivers is clearly below the nominal value of 0.6 m/mm (50 ft/in) of letter height. The means for older drivers are generally between 0.48 m/mm and 0.6 m/mm (40 and 50 ft/in); however, the 85th percentile values reported are between 0.36 and 0.48 m/mm (30 and 40 ft/in) (Sivak, Olson, and Pastalan, 1981; Kuemmel, 1992; Mace, Garvey, and Heckard, 1994). Mace (1988) concluded that a most conservative standard would provide drivers with 2 minutes of arc, which corresponds to 20/40 vision and a 0.36 m/mm (30 ft/in) LI.

In a laboratory simulation study, Staplin et al. (1990) found that older drivers (ages 65–80) demonstrated a need for larger letter sizes to discern a message on a guide sign, compared with a group of younger drivers (ages 19–49). To read a one-word sign, older drivers required a mean letter size corresponding to 2.5 minutes of visual angle (or a Snellen acuity of 20/50), compared with the mean size required by younger drivers of 1.8 minutes of visual angle (or Snellen acuity of 20/35). Character size requirements increased for both age groups when the message contained four words, to 3.78 minutes of visual angle (acuity equivalent of 20/75) for the older drivers, and to 2.7 minutes of visual angle (acuity equivalent of 20/54) for the younger drivers. The main effect of age for the word and message legibility measure was highly significant. Staplin et al. (1990) concluded that for standard highway signing, an increase in character size in the range of 30 percent appears necessary to accommodate age-related acuity differences across the driving population.

Finally, the MUTCD states that street-name signs should be placed at least on diagonally opposite corners so that they will be on the far right-hand side of the intersection for traffic on the major street. It further states that on intersection approaches, a supplemental street-name sign may be erected separately or below an intersection-related warning sign, and when combined with a yellow diamond sign, the color should be a black message on a yellow background. Burnham (1992) noted that signs located over the highway are more likely to be seen before those located on either side of the highway. In this regard, Zwahlen (1989) examined detection distances of objects in the peripheral field versus line-of-sight detection and found that average detection distances decrease considerably as the peripheral visual detection angle increases. Placement of street-name signs overhead places the sign in the driver's forward line of sight, eliminating the need for the driver to take his/her eyes away from the driving scene, and reduces the visual complexity of the sign's surround, but under some sky conditions (e.g., backlit by the sun at dawn and dusk) the sign may be unreadable. Thus, overhead street-name signing should be a supplement to standard roadside signing.

Use of a supplemental street-name sign with an advance warning crossroad, side road, or T-intersection sign (W2-1, W2-2, W2-3, and W2-4) provides the benefit of additional decision and maneuver time if a lane change is required prior to reaching the intersection. Midblock street-name signing provides the same benefit. Phoenix, Arizona, a city with a large older driver population, has been using "jumbo" street-name signs since 1973. These signs are 400 mm (16 in) in height and use 200-mm (8-in) capital letters.

K. Design Element: One-Way/Wrong-Way Signage

Table 14. Cross-references of related entries for one-way/wrong-way signage.

Applications in Standard Reference Manuals	
MUTCD (1988)	**AASHTO Green Book (1994)**
Pgs. 2A-12 - 2A-13, Sect. 2A-31 Pg. 2B-19, Sect. 2B-27 Pgs. 2E-22 - 2E-23, Sect. 2E-40	Pg. 726, Para. 4 Pg. 915, Para. 6

Vaswani (1974, 1977) found that approximately half of the incidents that involved wrong-way driving on multilane divided highways without access control occurred at intersections with freeway exits and with secondary roads. These wrong-way movements resulted from left-turning vehicles making a left turn into a lane on the near side of the median, rather than turning around the nose of the median into a lane on the far side. In an analysis of 96 accidents resulting from wrong-way movements on divided highways in Indiana from 1970 through 1972, Scifres and Loutzenheiser (1975) found that wrong-way movements most often occur under conditions of low traffic volume, low visibility, and low lane-use density. In addition, it was reported that 69 percent of the wrong-way drivers were drunk, older (age 65 and older), or fatigued (driving between 12 a.m. and 6 a.m.). A review of the literature by Crowley and Seguin (1986) reported that (1) there are significantly more incidents of wrong-way driving than there are accidents, and (2) drivers older than 60 years of age are overrepresented in wrong-way movements on a per-mile basis.

Further evidence of older driver difficulties likely to result in wrong-way movements was reported by McKnight and Urquijo (1993). These researchers examined 1,000 police forms that documented observations of incompetence when an older driver was either stopped for a violation or involved in a crash. They found that two of the primary behaviors that brought these drivers to the attention of police were driving the wrong way on a one-way street and driving on the wrong side of a two-way street. The drivers' mistakes contributed to many violations (149) but few accidents (29).

The ability to abstract information and make quick decisions about it are capabilities required to safely perform the driving task. Evidence has been found that older drivers' accidents often occur as the result of overly attending to irrelevant aspects of a driving scene (Planek and Fowler, 1971). Hasher and Zacks (1988) argued that older adults are deficient in inhibitory processes, and as a result, they frequently direct attention to irrelevant information at the expense of relevant information. The selective attention literature generally suggests that for adults of all ages, but particularly for the elderly, the most relevant information must be signaled in a dramatic manner to ensure that it receives a high priority for processing in situations where there is a great deal of complexity. Mace (1988) stated that age differences in glare sensitivity and restricted peripheral vision coupled with the process of selective attention may cause higher conspicuity thresholds for older drivers. Overall, these deficits point to the need for more effective and more conspicuous signs, realized through provision of multiple or advance signs as well as changes in size, luminance, or placement of signs.

The most comprehensive survey of current policies and practices for signing intersections to inform drivers of travel direction and to prevent wrong-way movements was conducted in the 48 contiguous States and in 35 of the largest cities by Crowley and Seguin (1986). They found considerable variability in the location, placement, and types of signs used to prevent wrong-way movements from occurring. The greatest variability in practice was reported in locations where a median divider exists. The study authors reported that median width is a key factor in the number, type, and location of signs to be used. When medians are extremely narrow, there appears to be little confusion that the intersecting roadway is two-way and drivers have less need for special signs to indicate travel direction. Where the median is sufficiently large, the intersection will be generally signed as two separate one-way roadways. A problem in defining what is "wide" and what is "narrow" was shown in the responses from a survey of practitioners across the United States, where there was a significant range in values around the 9 m (30 ft) delineation point specified by the MUTCD (para. 2A-31). The majority of jurisdictions tended to treat wide-median divided highways as if they were two separate intersections for the purpose of direction and turn-prohibition signing. The most commonly reported sign configuration implemented in the jurisdictions that responded to the survey was the MUTCD standard of a pair of ONE WAY signs (R6-1) on the near right-hand corners and far left-hand corners of *each* intersection with the directional roadway. A second pattern reported was a slight variation of the MUTCD standard, where the jurisdictions required a far-right sign (either a ONE WAY or a NO RIGHT TURN symbol sign) at the second intersection. Although many jurisdictions followed the MUTCD specifications for location of signs, many reported that they *replaced* a near-side ONE WAY sign with a NO RIGHT TURN sign (R3-1), even though the MUTCD states that the turn prohibition sign may be used to *supplement* the near-right/far-left pair of ONE WAY signs. The third pattern reported by some jurisdictions was to treat the divided highway, *regardless of median width*, as if it were a single intersection. In this case, a left/median sign for the first one-way roadway and a far-right sign for the second one-way roadway were considered sufficient. Where jurisdictions implement the third pattern, there was more emphasis on the use of the DIVIDED HIGHWAY CROSSING sign (R6-3) to supplement the limited amount of directional information. In one jurisdiction, signing was limited to the use of the DIVIDED HIGHWAY CROSSING sign.

Crowley and Seguin (1986) reported that some jurisdictions recommended the use of optional signs—i.e., DO NOT ENTER (R-5-1), WRONG WAY (R5-9), and KEEP RIGHT (R4-7)—but noted that these signs are not helpful to a motorist making decisions as he/she approaches an intersection; they are detected only when the driver begins a wrong-way movement upon reaching the intersection. In this regard, a number of jurisdictions reported that they required the use of the DIVIDED HIGHWAY CROSSING sign, as it is the only sign available that has a direct impact on the decision process of drivers approaching a divided highway with a median. The MUTCD states that this sign may be used as a supplemental sign on the approach legs of a roadway that intersects with a divided highway. Although this sign was not included in the set of traffic control devices tested by Hulbert and Fowler (1980), these researchers found that where complex driver judgments were required in conjunction with the use and understanding of particular driving situations, larger percentages of drivers failed to correctly respond to the meaning of traffic control devices. The comprehensibility of the DIVIDED HIGHWAY CROSSING sign has not been reliably documented.

Crowley and Seguin (1986) also conducted a laboratory study and a field validation study using subjects in three age groups (younger than age 25, ages 25–54, and age 55 and older) to identify signing practices that best provide information to minimize the possibility of wrong-way turning movements. Subjects were asked to identify driver actions that were either directly or by implication prohibited (by signs, markings, etc.), and to do so as quickly as possible. In the laboratory study, projected scenes of intersections containing a median (divided highway) were associated with higher error rates and longer decision latencies than scenes containing T-intersections and intersections of a two-way street with a one-way street (no median). The untreated intersections, where geometry alone was tested to determine the extent to which it conveyed an intrinsic "one-way" message, resulted in the worst performance; thus, any signing, regardless of the configuration, appears to be superior to no signing. However, even when the standard MUTCD near-right/far-left placement of ONE WAY signs were presented, large numbers of subjects did not recognize that the projected scene was that of a divided highway. Furthermore, the addition of a DIVIDED HIGHWAY CROSSING sign at the near-right corner of the intersection did not significantly reduce the overall error rate. Subjects age 55 and older had fewer correct responses and longer decision latencies than subjects in the two younger age groups. Field study results showed the following: (1) unsignalized divided highways resulted in more extreme steering patterns than signalized divided highways, at both of the one-way locations; (2) the use of ONE WAY signs in the left/median and far-right locations for medians as narrow as 6 m (20 ft) and as wide as 12.8 m (42 ft) showed superior performance to the single left/median ONE WAY sign; and (3) at undivided intersections of a two-way street with a one-way street, the most extreme variation in steering position was shown for the untreated intersections, suggesting that any signing treatment is better than none.

Crowley and Seguin (1986) noted that because there are intersections with specific physical factors that make the basic near-right/far-left rule inappropriate, the following text should be added to the MUTCD in section 2B-29 to bring the MUTCD and actual practice more in agreement and to reflect the actual manner in which the practitioner must respond to the problem of signing to prevent wrong-way traffic movements while providing positive guidance to drivers: "However, if an engineering study demonstrates the specified placements to be inappropriate due to factors such as sight distance restrictions, approach roadway grade and/or alignment, complex background, etc., one-way signs should be placed so as to provide the best possible guidance for the driver." In addition, a revision to section 2A-31 was proposed, which states that for medians of 9 m (30 ft) and under, both the left/median and far-right locations should be implemented when a divided highway justifies any form of one-way signing (see figure 7). DIVIDED HIGHWAY CROSSING, DO NOT ENTER, and WRONG WAY signs are optional, depending on the specific problem at a narrow median intersection. The authors note, however, that when a median is very narrow, one-way signing is usually unnecessary.

For medians greater than 9 m (30 ft), Crowley and Seguin (1986) suggested the use of ONE WAY signs posted at each of the following locations, for each direction of traffic: near right, median left, and far right. WRONG WAY and DO NOT ENTER signs are again optional. The resulting configuration is consistent with that shown earlier in Recommendation 4 of Design Element E.

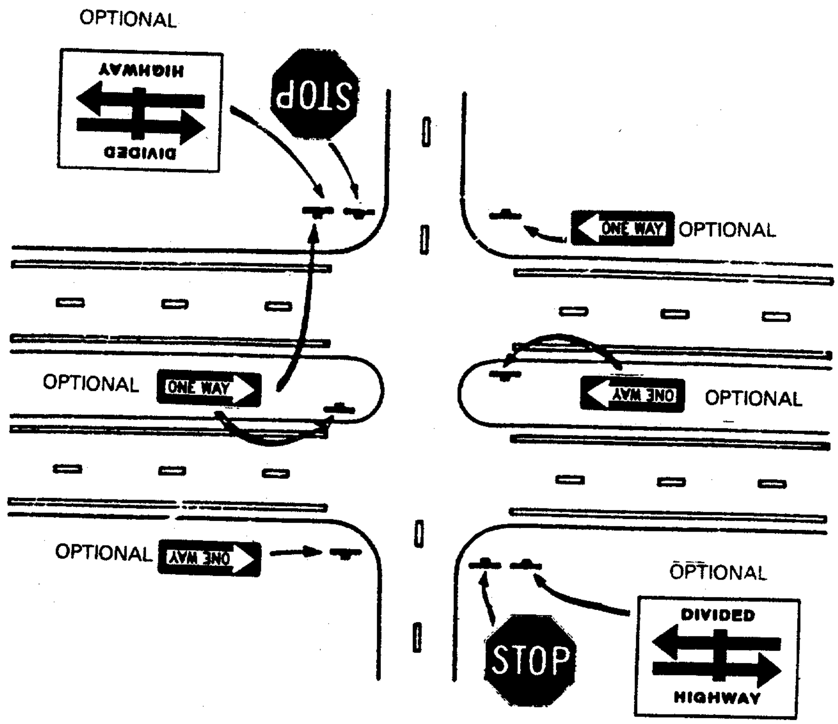


Figure 7. Suggested revision of MUTCD Figure 2-3a, for medians less than or equal to 9 m (30 ft). Source: Crowley and Seguin, 1986.

For T-intersections, Crowley and Seguin (1986) recommended that near-right side ONE WAY signs and far side ONE WAY signs be located so that drivers are most likely to see them *before* they begin to make a wrong-way movement. The optimal placement for the far side sign would be opposite the extended centerline of the approach leg as shown in MUTCD figure 2-4. However, where a study indicates that the far-side centerline location is not appropriate at a particular intersection because of blockage, distracting far-side land use, excessively wide approach leg, etc., these authors suggested that the best alternate location is the far left-hand corner for one-way traffic moving from left to right, and the far right-hand corner for traffic moving from right to left (see figure 8).

For four-way intersections (i.e., the intersection of a one-way street with a two-way street), the near-right/far-left locations were recommended by Crowley and Seguin (1986) regardless of whether there is left-to-right or right-to-left traffic. An additional ONE WAY sign located on the far-right side may be necessary in certain locations where approach grade and angle may direct the driver's field of view away from the "normal" sign locations (see figure 9).

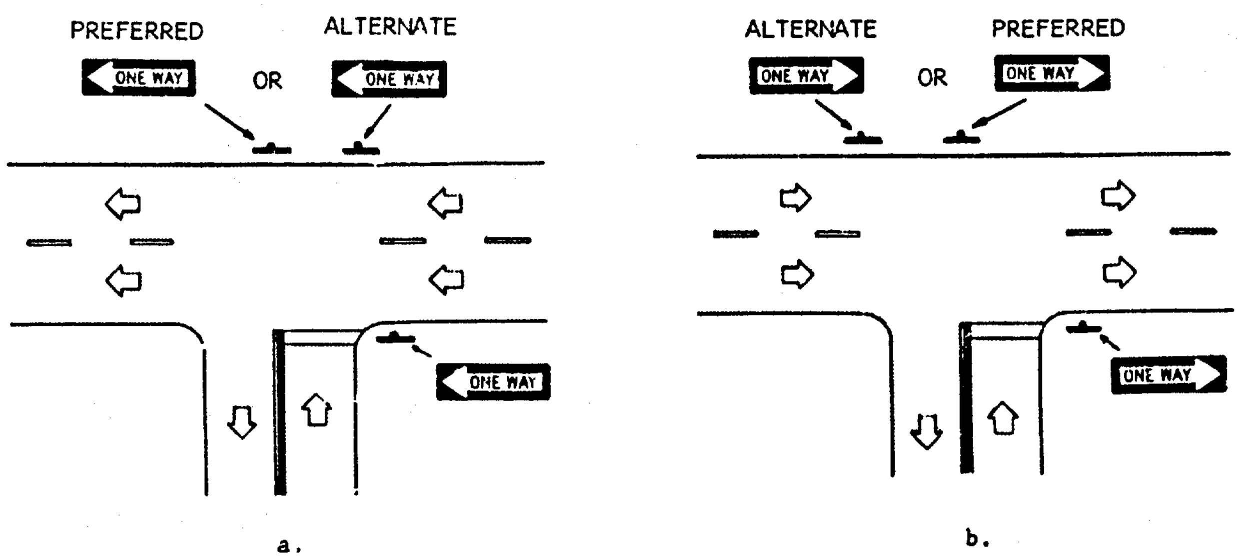


Figure 8. Recommended location of ONE WAY signs for T-type intersections. Source: Crowley and Seguin, 1986.

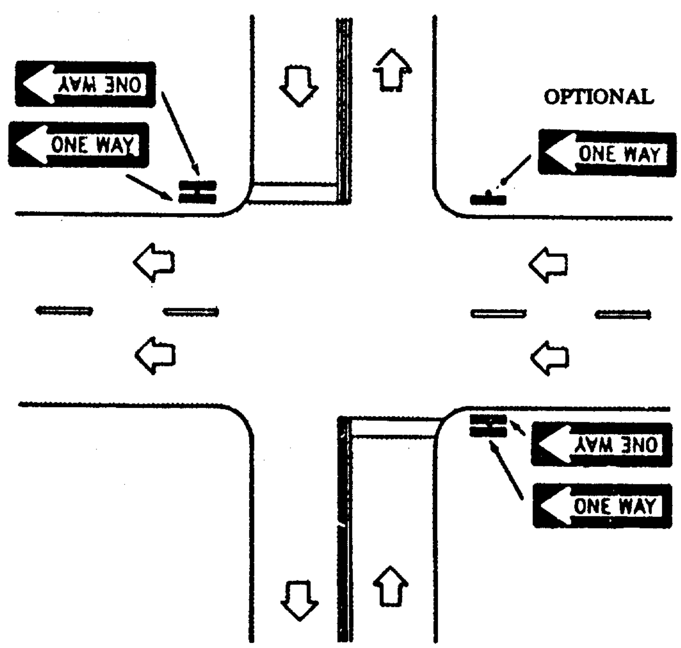


Figure 9. Recommended location of ONE WAY signs for intersection of a one-way and two-way street. Source: Crowley and Seguin, 1986.

Finally, as noted in the "Rationale and Supporting Evidence" for Design Element E, the potential for wrong-way movements at intersections with channelized (positive) offset left-turn lanes (within a raised median) increases for the driver turning left from the minor road onto the major road, who must correctly identify the proper median opening into which he/she should turn. The following countermeasures were recommended at intersections with a divided median on the receiving leg, where the left-turn lane treatment results in channelized offset left-turn lanes (e.g., a parallel or tapered left-turn lane between two medians); these countermeasures are intended to reduce the potential for wrong-way maneuvers by drivers turning left from the stop-controlled minor roadway:

- Proper signing (advance DIVIDED HIGHWAY CROSSING signs, and proper positioning of WRONG WAY, DO NOT ENTER, and ONE WAY signing at the intersection) must be implemented.

- The channelized left-turn lanes should contain white painted lane-use arrow pavement markings (left-turn only).

- Pavement markings which scribe a path through the turn are recommended to reduce the likelihood for the wrong-way movement.

- The use of a wide (600-mm [24-in]) white stop bar is recommended at the end of the channelized left-turn lane as a countermeasure to aid in preventing a potential wrong-way movement. This countermeasure was found to be effective in preventing wrong-way entries onto freeway exit ramps in Georgia (Parsonson and Marks, 1979).

- Placement of 7-m (23.5-ft) wrong-way arrows in the through lanes is recommended, as specified in the MUTCD requirements for wrong-way traffic control for locations determined to have a special need, section 2E-40. Wrong-way arrows have been shown to reduce the frequency of wrong-way movements at freeway interchanges (Parsonson and Marks, 1979).

- Indistinct medians are considered to be design elements that tend to reduce a driver's ability to see and understand the overall physical and operational features of an intersection, increasing the frequency of wrong-way movements (Scifres and Loutzenheiser, 1975). Delineation of the median noses using reflectorized paint and other treatments will increase their visibility and should improve driver understanding of the intersection design and function.

The recommended placement of these traffic control devices was illustrated in Recommendation 4 of Design Element E.

L. Design Element: Stop- and Yield-Controlled Intersection Signage

Table 15. Cross-references of related entries for stop- and yield-controlled intersection signage.

Applications in Standard Reference Manuals	
MUTCD (1988)	AASHTO Green Book (1994)
Pg. 2B-2, Sect. 2B-4 Pg. 2B-3–2B-6, Sect(s). 2B-6 - 2B-9 Pg. 2C-8, Sect(s). 2C-15 & 2C-16	Pgs. 700-703, Sect(s). on *Yield Control for Minor Roads* and *Stop Control for Minor Roads* Pg. 919, Sect on *At-Grade Terminals*

Drivers approaching a nonsignalized intersection must be able to detect the presence of the intersection and then detect, recognize, and respond to the intersection traffic control devices present at the intersection. Next, drivers must detect potential conflict vehicles, pedestrian crosswalk locations, and pedestrians at or near the intersection. Proper allocation of attention has become more difficult, as drivers are overloaded with more traffic, more signs, and more complex roadway configurations and traffic patterns, as well as more complex displays and controls in newer vehicles (Dewar, 1992). The presence of large commercial signs near intersections has been associated with a significant increase in accidents at stop-controlled intersections (Holahan, 1977).

Mace and Pollack (1983) noted that conspicuity is not an observable characteristic of a sign but a construct which relates measures of perceptual performance with measures of background, motivation, and driver uncertainty. In this regard, conspicuity may be aided by multiple treatments or advance signing as well as changes in size, contrast, and placement. They noted that STOP signs following a STOP AHEAD (W3-1a) sign are more conspicuous not only to older drivers but to everyone, because expectancy has been increased.

The need for appropriate levels of brightness to ensure conspicuity and timely detection by drivers of highway signs, including STOP and YIELD signs, was addressed in FHWA-sponsored research to establish minimum retroreflectivity requirements for these devices. Mace developed a model to derive the retroreflectivity levels necessary for adequate visibility distance, taking into account driver age and visual performance level, as well as the driver's response requirements (action versus no action) to the information presented on a given sign when encountered in a given situation (city, highway) with an assumed operating speed (ranging from 16 km/h [10 mi/h] to 104 km/h [65 mi/h]), for signs of varying size and placement (shoulder, overhead). This work is reported by Ziskind, Mace, Staplin, Sim, and Lococo (1991), and subsequent guidelines have been promulgated by FHWA. Taking speed and sign application into account, the recommended retroreflectivity for STOP signs resulting from this research ranged between 10 cd/m^2/lux up to 24 cd/m^2/lux for the sign background (red) area, with significantly higher values for the sign legend. For the YIELD sign, the recommended levels ranged between 24 and 39 cd/m^2/lux. These units express the sign brightness or luminance, measured in candelas per square meter, resulting from a given level of incident illumination, measured in lux.

A retroreflectometer is used to obtain these data in the field. Because both the STOP and YIELD signs are so extensively overlearned by drivers, their comprehension is believed to be associated with the icon, i.e., their unique shape and coloration. Thus, the brightness of the sign's background area is most critical, because these devices will typically be recognized and understood as soon as they are detected.

Age-related deficits in vision and attention are key to developing recommendations for improved stop control and yield control at intersections. Researchers examining the State accident records of 53 older drivers found that those with restrictions in their useful field of view (UFOV)—a measure of selective attention and speed of visual processing—had 15 times more intersection accidents than those with normal visual attention (Owsley, Ball, Sloane, Roenker, and Bruni, 1991). A follow-up study with a sample of 300 drivers demonstrated that UFOV could account for up to 30 percent of the variance in intersection accident experience (Ball, Owsley, Sloane, Roenker, and Bruni, 1994). Additional relevant findings may be cited from a simulator study of peripheral visual field loss and driving impairment which also examined the actual driving records of the study participants (Szlyk, Severing, and Fishman, 1991). It was found that visual function factors, including acuity as well as visual field measures, could account for 26 percent of the variance in real-world accidents. Also, greater visual field loss was associated in the simulator data with greater distance traveled ("reaction distance") before responding to a peripheral stimulus (e.g., a STOP sign).

A considerable body of evidence exists documenting the difficulties of older driver populations in negotiating stop-controlled intersections. Specifically, analyses of accident and violation types at these sites highlight the older driver's difficulty in detecting, comprehending, and responding to signs within an appropriate timeframe for the safe completion of intersection maneuvers.

Statistics on Iowa fatal accidents show that during 1986–1990, running STOP signs was a contributing circumstance in 297 fatal accidents which killed 352 people; drivers age 65 and older accounted for 28 percent of the fatal crashes, and drivers younger than age 25 were involved in 27 percent of the fatal crashes (Iowa Department of Transportation, 1991). Stamatiadis, Taylor, and McKelvey (1991) found that at stop-controlled urban intersections, the percentage of drivers age 75 and older involved in right-angle accidents was more than double that of urban signalized intersections. Malfetti and Winter (1987), reporting on the unsafe driving performance of drivers age 55 and older, noted that older drivers frequently failed to respond properly or respond at all to road signs and signals; descriptions of their behavior included running red lights or STOP signs and rolling through STOP signs. Some older persons' behavior at STOP signs and signals seemed to indicate that they did not understand why they needed to wait when no other traffic was coming. Brainan (1980) used in-car observation to gain firsthand knowledge and insight into older people's driving behavior. Drivers in the 70 and older age group showed difficulty at two of the STOP signs on the test route; their errors were in failing to make complete stops, poor vehicle positioning at STOP signs, and jerky and abrupt stops. Campbell, Wolfe, Blower, Waller, Massie, and Ridella (1990), looking at police reports of crossing accidents at nonsignalized intersections, found that older drivers often stopped and then pulled out in front of oncoming traffic, whereas younger drivers more often failed to stop at all. Further evidence of unsafe behaviors by older drivers was provided in a study by McKnight and Urquijo (1993). Their data consisted of 1,000 police referral forms from the motor vehicle departments of California, Maryland, Massachusetts, Michigan, and Oregon; the forms included observations of incompetent behavior exhibited by older drivers who were stopped for a violation by law enforcement personnel or were involved in an accident. The specific behaviors contributing to the contact between the older driver and the police officer included failing to yield right-of-way or come to a complete stop at a STOP sign, and failing to stop or yield to other traffic; taken together, these behaviors contributed to significant numbers of accidents (74) and violations (114).

Data from 124,000 two-vehicle accidents (54,000 accidents at signalized intersections and 70,000 accidents at nonsignalized intersections) showed that drivers younger than age 25 and older than age 65 were overinvolved in accidents at both types of intersections (Stamatiadis et al. 1991). However, the overinvolvement of older drivers in *nonsignalized* intersection accidents was more pronounced than it was for signalized intersection accidents. Although the total number of accidents was reduced at nonsignalized intersections that contained signs when compared with unsigned intersections, the accident involvement ratios of older drivers were higher at signed intersections than at unsigned intersections. At nonsignalized intersections, the highest percentage of fatalities were the result of right-angle collisions (25 percent). In terms of the frequency of injury at nonsignalized intersections, rear-end accidents were the most frequent cause (35 percent), followed by right-angle accidents (18 percent), other-angle accidents (10 percent), and head-on/left-turn accidents (8 percent). The leading violation types for all older drivers in descending order were failure to yield right-of-way, following too closely, improper lane usage, and improper turning. At nonsignalized intersections, older drivers showed the highest accident frequency on major streets with two lanes in both directions (a condition most frequently associated with high-speed, low-volume rural roads), followed by roads with four lanes, and those with five lanes in both directions. These configurations were most often associated with low-speed, high-volume urban locations, where intersection negotiation involves more complex decisions involving more conflict vehicles and more visually distracting conditions.

Cooper (1990) utilized a database of all 1986 police-attended accidents in British Columbia, in an effort to compare the crash characteristics of older drivers with those of their younger counterparts. While 66.5 percent of crashes involving drivers ages 36–50 occurred at intersections, the percentage increased to 69.2 percent, 70.7 percent, and 76.0 percent for drivers ages 55–64, 65–74, and 75 and older, respectively. Overall, the two oldest groups identified in this analysis were significantly more accident involved at STOP/YIELD sign locations and less involved at either uncontrolled or signal-regulated locations. In follow-on questionnaires administered to a sample of drivers in each age group studied, intersection negotiation was mentioned by the older drivers as second in difficulty to problems changing lanes. About 20 percent of the older drivers mentioned not stopping properly at STOP signs. Vehicle maneuvering prior to the accident was a key variable for drivers over age 65, and in particular, for left turns at uncontrolled or STOP/YIELD sign-controlled intersections. Drivers ages 36–50 experienced only 10.9 percent of their accidents while turning left at this type of intersection, compared with 13.0, 15.4, and 19.5 percent of drivers ages 55–64, 65–74, and 75 and older, respectively.

Council and Zegeer (1992) conducted an analysis of intersection accidents occurring in Minnesota and Illinois for the time period of 1985–1987 to highlight accident types, situations, and causes of accidents, in an effort to increase the knowledge of how older drivers react at intersections. For all the analyses, comparisons were made between a "young-old" group (ages 65–74), an "old-old" group (age 75 or older), and a "middle-aged" comparison group (ages 30–50). Their findings regarding driver age differences in collision types, pre-accident maneuvers, and contributing factors are described below.

With respect to collision type at stop-controlled intersections, analysis of the data showed little difference in the proportion of crashes involving left-turning vehicles at either urban or rural locations when the older groups were compared with the middle-aged group. There was, however, a significant overinvolvement for both groups of older drivers in right-angle collisions, both in urban and in rural locations. At urban intersections, right-angle collisions accounted for 56.1 percent of the middle-aged driver accidents, compared with 64.7 percent of the young-old, and 68.3 percent of the old-old driver accidents. These percentages increase for all groups at rural intersections—61.3, 68.6, and 71.2 percent, respectively for middle-aged drivers, young-old drivers, and old-old drivers. Data for yield-controlled intersections showed older drivers overcontributing to left-turn collisions in urban areas and to angle collisions in both urban and rural areas.

Regarding pre-accident maneuvers at stop-controlled intersections, for both rural and urban locations, right-angle collisions were the most frequent collisions, and middle-aged drivers were more likely to be traveling straight or slowing/stopping than the two older groups. The older drivers were more likely to be turning left or starting from a stop than their younger counterparts. The pattern is similar for left-turning crashes. For rear-end collisions, the old-old drivers were more likely to be going straight (thus being the striking vehicle), and the middle-aged and young-old drivers were more likely to be stopped or slowing. For the few right-turning collisions at urban stop-controlled intersections, the middle-aged drivers were going straight and the old-old drivers were more likely to be turning left or right or starting from a stop. Rural stop-controlled locations showed the same patterns of precrash maneuvers among the three age groups.

Finally, breakdowns of contributing factors for the urban and rural stop-controlled intersections showed that the middle-aged drivers exhibited a higher proportion of no improper driving behavior, while the young-old and old-old drivers were more often cited for failure-to-yield, disregarding the STOP sign, and driver inattention. When starting from a stop, however, the old-old drivers had a *lower* probability of being cited for improper driving. When cited, the old-old group was more likely to have disregarded the STOP sign than the other two driver groups. The young-old drivers as well as the old-old drivers more frequently failed to yield than the middle-aged drivers.

For left turns, the middle-aged drivers again were more frequently found to have exhibited "no improper driving." The two older driver groups were most frequently cited with failure-to-yield. There was no difference in the number of drivers in each age group who disregarded the STOP sign. For going-straight situations, the middle-aged driver was found to have exhibited no improper driving behavior twice as often as the young-old driver and almost three times as often as the old-old driver. Failing to yield, disregarding the STOP sign, and inattention were most often cited as the contributing factor for the two older groups.

A two-way stop requires a driver to cross traffic streams from either direction; this poses a potential risk, because cross traffic may be proceeding rapidly and drivers may be less prepared to accommodate to errors made by crossing or turning drivers. Most critically, drivers proceeding straight through the intersection must be aware of the fact that the cross-street traffic does not stop, and that they must yield to cross-street vehicles from each direction before proceeding through the intersection. Older drivers are disproportionately penalized by the late realization of this operating condition, due to the various sources of response slowing noted earlier. Studies of cross-traffic signing to address this problem have shown qualified but promising results in a number of jurisdictions (Gattis, 1996). Although findings indicate that conversion of two-way to four-way stop operations may be more effective in reducing intersection accidents than the use of cross-traffic signing, there are obvious tradeoffs for capacity from this strategy. However, data from accident analyses in Arkansas, Oregon, and Florida reported by Gattis (1996) showed significant reductions in right-angle crashes after cross-traffic signing was installed at problem intersections. At this time there is no standard sign design to convey this message; Ligon, Carter, and McGee (1985) identified a number of alternate wordings used in different States. In addition, a warrant for use of a cross-traffic sign applied in the State of Illinois may be reviewed in the Gattis (1996) article.

The issue of driver expectancy, a key predictor of performance for older motorists, was addressed in a study by Agent (1979) to determine what treatments would make drivers more aware of a stop-ahead situation. Agent concluded that at rural sites, transverse pavement striping should be applied approximately 366 m (1,200 ft) in advance of the STOP sign to significantly reduce approach speeds. Later research (Agent, 1988) recommended the following operational improvements at intersections controlled by STOP signs: (1) installing additional advance warning signs; (2) modifying warning signs to provide additional notice; (3) adding stop bars to inform motorists of the proper location to stop, to obtain the maximum available sight distance; (4) installing rumble strips, transverse stripes, or post delineators on the stop approach to warn drivers that they would be required to stop; and (5) installing beacons. Although Agent emphasized that beacons do not eliminate the problem of drivers who disregard the STOP sign, flashing beacons used in conjunction with STOP signs at isolated intersections or intersections with restricted sight distance have been consistently shown to be effective in decreasing accidents by increasing driver awareness and decreasing approach speeds (California Department of Public Works, 1967; Cribbins and Walton, 1970; Goldblatt, 1977; King, Abramson, Cohen, and Wilkinson, 1978; Lyles, 1980).

With regard to the accident reduction effectiveness of rumble strips placed on intersection approaches, Harwood (1993) reported that rumble strips can provide a reduction of at least 50 percent in the types of accidents most susceptible to correction, including accidents involving running through a STOP sign. They can also be expected to reduce vehicle speed on intersection approaches and to increase driver compliance with STOP signs. In an evaluation conducted by the Virginia Department of Highways and Transportation (1981a) where rumble strips were installed at stop-controlled intersections, the total accident frequency was reduced by 37 percent, fatal accidents were reduced by 93 percent, injury accidents were reduced by 37 percent, and property-damage-only accidents were reduced by 25 percent. In this study, 39 of the 141 accidents in the before period were classified as being types susceptible to correction by rumble strip installation, particularly rear-end accidents and ran-STOP-sign accidents. The accident rate for these accident types was reduced by 89 percent. Carstens and Woo (1982) found that primary highway intersections where rumble strips were installed experienced a statistically significant reduction in the accident rate in the first year or two following their installation, both at four-way and T-intersections. The accident rate at the 21 study intersections decreased by 51 percent for total accidents and by 38 percent for ran-STOP-sign accidents. Carstens and Woo found no statistically significant change in accident rate at 88 intersections on secondary roads where rumble strips were installed. They concluded that rumble strips are more effective at primary highway intersections than secondary road intersections for the following reasons: (1) primary highways serve a higher proportion of drivers who are unfamiliar with the highway; (2) trips tend to be longer on primary highways so that fatigue and the monotony of driving may play a more important role than on secondary roads; (3) traffic volumes are higher on primary highways, so the number of potential conflicts is greater; and (4) the geometric layout of primary highway intersections is often more complex than that of secondary road intersections. These researchers also found that rumble strips may be more effective in reducing nighttime accidents at unlighted intersections than at lighted intersections. Harwood (1993) reported that several highway agencies commented that it was important to avoid the temptation to use rumble strips where they are not needed; if every intersection had rumble strips on its approach, rumble strips would soon lose their ability to focus the attention of the motorist on an unexpected hazard.

Before concluding this discussion, certain aspects of YIELD sign operations deserve mention. A YIELD sign facilitates traffic flow by preventing unnecessary stops and allowing drivers to enter the traffic flow with minimum disruption of through traffic. Most YIELD signs are posted where right-turning drivers can approach the cross street at an oblique angle. Such configurations benefit elderly drivers in carrying out the turning maneuver by avoiding the tight radii that characterize right-angle turns. However, in several respects, intersections regulated by YIELD signs place greater demands upon drivers than those employing other controls, in terms of gap selection, difficulty with head turning, lanekeeping, and maintaining or adjusting vehicle speed. The angle of approach to the street or highway being entered ranges from the near perpendicular to the near parallel. The closer the angle is to the parallel, the further the driver must turn his/her head to detect and to judge the speed and distance of vehicles on the road to be entered. Many elderly drivers are unable to turn their heads far enough to get a good look at approaching traffic, while the need to share attention with the road ahead necessarily limits the lane exposure to 1 or 2 s. Some drivers are reduced to attempting to judge distance and gaps by means of the outside mirror. The inability to judge gaps in this manner often results in the driver reaching the end of the access lane without having identified an appropriate gap. The driver in this situation comes to a complete stop and then must enter the cross street by accelerating from a stopped position. The difficulty in judging gaps may lead to aborted attempts to enter the roadway, leaving the older driver vulnerable to following drivers who direct their attention upstream and fail to notice that a vehicle has stopped in front of them. The need to share attention between two widely separated points results in eyes being off the intended path for lengthy periods. The diversion of attention, along with movement of the upper torso, hampers the older driver's ability to maintain directional control.

McGee and Blankenship (1989) report that intersections converted from stop to yield control are likely to experience an increase in accidents, especially at higher traffic volumes, at the rate of one additional accident every 2 years. In addition, converted yield-controlled intersections have a higher accident rate than established yield-controlled intersections. They note that while yield control has been found to be as safe as stop control at very low volumes, the safety impacts are not well established for higher volume levels. Agent and Deen (1975) reported that rural road accident types at yield-controlled intersections are different from those at stop-controlled intersections. At YIELD signs, more than half of the accidents were rear-end collisions, while more than half of the accidents at STOP signs were angle collisions.

M. Design Element: Devices for Lane Assignment on Intersection Approach

Table 16. Cross-references of related entries for devices for lane assignment on intersection approach.

Applications in Standard Reference Manuals			
MUTCD (1988)	**AASHTO Green Book (1994)**	**Roadway Lighting Handbook Chapter 6 (1983)**	**NCHRP 279 Intersection Channelization Design Guide (1985)**
Pgs. 2B-11 - 2B-12, Sect. 2B-17 Pg. 2B-13, Para. 1 Pg. 3A-1, Sect(s). 3A-1 & 3A-2 Pgs. 3A-2 - 3A-4, Sect(s). 3A-5 - 3A-7 Pgs. 3B-1 - 3B-2, Sect(s). 3B-1 & 3B-2 Pgs. 3B-8 - 3B-21, Sect(s). 3B-6 - 3B-15 Pg. 3B-27, Para(s) 1-5 Pgs. 3B-29 - 3B-30, Fig(s). 3-18 & 3-19	Pg. 250, Para. 3 Pg. 314, Para. 7 Pg. 343, Para. 2 Pg. 490, Para. 4 Pg. 517, Para. 6 Pg. 534, Para. 4 Pgs. 629-641, Sect. on *Three-Leg Intersections*, *Channelized Three-Leg Intersections*, *Four-Leg Intersections*, and *Channelized Four-Leg Intersections* Pg. 637, Para. 7 Pg. 739, Para. 3 Pg. 740, Para(s). 4-5 Pg. 741, Para. 2 through Table IX-15 on Pg. 743 Pgs. 749-751, Sect. on *Speed-Change Lanes at Intersections* Pgs. 778-792, Sect(s). on *Auxiliary Lanes*, *Simultaneous Left Turns*, *Intersection Design Elements with Frontage Roads*, and *Bicycles at Intersections* Pgs. 781-786, Sect(s). on *Taper* and *Median Left-Turn Lanes* Pg. 927, Para. 3 Pg. 929, Para(s). 6 & 9 Pgs. 933-934, Fig(s). X-68 and X-70 Pg. 936, Item 8	Pg. 25, Top two fig(s). and bottom left fig.	Pg. 1, Item 2, 3rd Bullet Pg. 19, Middle Fig. Pg. 24, Para. 1 and Top Fig. Pg. 34, Para. 1 & Top Fig. Pgs. 49 & 51, Sect(s). on *New Construction - Unsignalized Intersections* and *Reconstruction/Rehabilitation* and Fig. 4-12 Pg. 59, Fig. 4-20

As a driver approaches an intersection with the intention of traveling straight through, turning left, or turning right, he/she must first determine whether the currently traveled lane is the proper one for executing the intended maneuver. This understanding of the downstream intersection geometry is accomplished by the driver's visual search and successful detection, recognition, and comprehension of pavement markings (including stripes, symbols, and verbal pavement markings); regulatory and/or advisory signs mounted overhead, in the median, and/or on the shoulder in advance of the intersection; and other geometric feature cues such as curb and pavement edge lines, pavement width transitions, and surface texture differences connoting shoulder or median areas. Uncertainty about downstream lane assignment produces hesitancy during the intersection approach; this in turn decreases available maneuver time and diminishes the driver's attentional resources available for effective response to potential traffic conflicts at and near intersections.

Older drivers' decreased contrast sensitivity, reduced useful field of view, increased decision time—particularly in response to unexpected events—and slower vehicle control during movement execution combine to put these highway users at greater accident risk when approaching and negotiating intersections. Contrast sensitivity and visual acuity are the visual/perceptual requirements necessary to detect pavement markings and symbols and to read lane control signs and verbal and symbolic pavement markings. The early detection of lane control devices, by cueing the driver in advance that designated lanes exist for turning and through maneuvers, promotes safer and more confident performance of any required lane changes. This is because the traffic density is lighter, there are more available gaps, and there are fewer potential conflicts with other vehicles and pedestrians the farther away from the intersection the maneuver is performed. Of course, even the brightest delineation and pavement markings will not be visible to an operator unless an adequate sight distance (determined by horizontal and vertical alignment) is available.

In an effort to analyze the needs and concerns of older drivers, the Illinois Department of Transportation sponsored a statewide survey of 664 drivers, followed up by focus group meetings held in rural and urban areas (Benekohal, Resende, Shim, Michaels, and Weeks, 1992). Within this sample, the following four age categories were used for statistical analyses: ages 66–68, ages 69–72, ages 73–76, and age 77 and older. Comparisons of responses from drivers ages 66–68 and age 77 and older showed that the older group had more difficulty following pavement markings, finding the beginning of the left-turn lane, driving across intersections, and driving during daytime. Similarly, the level of difficulty for reading street signs and making left turns at intersections increased with increasing driver age. Turning left at intersections was perceived as a complex driving task, made more difficult when channelization providing visual cues was absent and only pavement markings designated which lane ahead was a through lane and which was a turning lane. The processes of lane location, detection, and selection must be made upstream at a distance where a lane change can be performed safely. Late detection by older drivers will result in erratic maneuvers such as lane weaving close to the intersection (McKnight and Stewart, 1990).

More than half of the 81 older drivers participating in more recent focus group discussions stated that quite often they suddenly find themselves in the wrong lane, because (1) they have certain expectations about lane use derived from intersections encountered earlier on the same roadway, (2) the advance signing is inadequate or lacking, or (3) the pavement markings are covered by cars at the intersection (Staplin, Harkey, Lococo, and Tarawneh, 1997). The biggest problem with turn-only lanes reported by group participants was that there is not enough warning for this feature. The appropriate amount of advance notice, as specified by these drivers, ranged from 5 car lengths to 1.6 km (1 mi). Sixty-four percent of the participants said that multiple warning signs are necessary when the right lane becomes a turn-only lane, with the need for an initial sign 20 to 30 s away, and a second sign 10 s away from the turn location. The remaining participants said that these distances should be increased.

Even greater consensus was shown in this study regarding sign location for lane assignment. Seventy-nine percent of the group reported that overhead lane-use signs are far more effective than roadside-mounted signs for this type of warning. Several participants suggested that a combination of roadside and overhead signs, in addition to painted roadway markings, would be beneficial. Although painted roadway markings were deemed helpful, 84 percent of all participants stated that they are useless in isolation from signs, because they are usually *at* the intersection and are obscured by traffic, and they are frequently worn and faded. The result is that drivers end up in the wrong lane and must go in a direction they had not planned for, or they try to change lanes at a point where it is not safe to do so. Thus, a general conclusion from this study is that overhead signing posted in advance of, as well as at, an intersection provides the most useful information to drivers about movement regulations which may be difficult to obtain from painted arrows when traffic density is high or when pavement markings are obscured by snow or become faded, or where sight distance is limited.

In an early study conducted by Hoffman (1969), the installation of overhead lane-use control signs in advance of six intersections in Michigan contributed to a reduction in the total number of accidents by 44 percent in a 1-year period, and a reduction in the incidence of accidents caused by turning from the wrong lane by 58 percent. More recently, older drivers (as well as their younger counterparts) have been shown to benefit from redundant signing (Staplin and Fisk, 1991). In addition to redundant information about right-of-way movements at intersections, drivers should be forewarned about lane drops, shifts, and merges through advance warning signs, and ideally these conditions should not occur close to an intersection. Advance route or street signing as well as reassurance (confirmatory) signing/route marker assemblies across the intersection will aid drivers of all ages in deciding which lane will lead them to their destination, prior to reaching the intersection.

The MUTCD (sections 2B-17 and 2B-18) states that the standard size of lane-use control signs (R3-5 through R3-8) shall be 750 x 900 mm (30 x 36 in) when post-mounted, and when post-mounted lane-use control signs are used, one sign should be placed *at* the intersection and a second sign placed an adequate distance *in advance* of the intersection so that motorists can select the appropriate lane before waiting to reach the end of the line of waiting vehicles. It also states that overhead lane-use control signs are preferred because they can be placed over the lanes to which they apply. Section 2A-16 states that overhead sign installations should be illuminated where an engineering study shows that reflectorization will not perform effectively. The MUTCD further states that pavement markings may be used to supplement post-mounted signs and should be used with mandatory turn signs. With regard to pavement word and symbol markings, section 3B-20 specifies that large letters and numerals (2.4 m [8 ft] or more in height) should be used and that markings should be repeated in advance of mandatory turn lanes when necessary to prevent entrapment and to help motorists select the appropriate lane before reaching the end of the line of waiting vehicles. Although pavement markings have obvious limitations (e.g., limited durability when installed in areas exposed to heavy traffic, poor visibility on wet roads, and obscuration by snow in some regions), they have the advantage of presenting information to drivers without distracting their attention from the roadway.

Finally, the Institute of Transportation Engineers identified several features to enhance the operation of urban arterial trap lanes (through lanes that terminate in an unshadowed mandatory left- or right-turn regulation): (1) signing that gives prominent advance notice of the unexpected mandatory turn regulation, followed by a regulatory sign at the point where the mandatory turn regulation takes effect, followed by a third sign at the intersection itself if there are intervening driveways from which motorists might enter the lane; (2) supplemental pavement markings which consist of a double-width broken lane line beginning at the advance warning sign and extending to the first regulatory sign; (3) a pavement legend in the trap lane; and (4) overhead signing. Candidates for these remediations include left-turn trap lanes on roadways with high volumes, high speeds, poor approach visibility, and complex geometrics (Foxen, 1986).

N. Design Element: Traffic Signal Performance Issues

Table 17. Cross-references of related entries for traffic signal performance issues.

Applications in Standard Reference Manuals	
MUTCD (1988)	AASHTO Green Book (1994)
Pgs. 4B-6 - 4B-7, Sect(s). 4B-7 & 4B-8 Pgs. 4B-10–4B-14, Sect(s). 4B-10–4B-12 Pg. 4b-15, Sect. 4B-14 Pgs. 4B-20–4B-21. Sect. 4B-28	Pgs. 318-319, Para. 1 of *Signal* Section Pgs. 480-481, Sect. on *Traffic Control Devices* Pgs. 534-535, Sect. on *Traffic Control Devices* Pg. 637, Para. 7 Pg. 739, Para(s). 4-5 Pg. 939, Para. 3 Pg. 534, Para. 4

Traffic signals are power-operated signal displays used to regulate or warn traffic. They include displays for intersection control, flashing beacons, lane-directional signals, ramp-metering signals, pedestrian signals, railroad-crossing signals, and similar devices. Warrants for traffic signals are thoroughly described in the MUTCD. The decision to install a traffic signal is based on an investigation of physical and traffic flow conditions and data, including traffic volume, approach travel speeds, physical condition diagrams, accident history, and gap and delay information (Wilshire, 1992). The MUTCD incorporates the intensity, light distribution, and chromaticity standards from the following Institute of Transportation Engineers (ITE) standards for traffic signals: *Vehicle Control Signal Heads*, ITE Standard No ST-008B (ITE, 1985b); *Pedestrian Traffic Control Signal Indications*, ITE Standard No. ST-011B (ITE, 1985a); *Traffic Signal Lamps*, ITE Standard No. ST-010 (ITE, 1986); and *Lane-Use Traffic Control Signal Heads* (ITE, 1980). Standards for traffic signals are important because it is imperative that they attract the attention of virtually every driver, including older drivers and those with impaired vision who meet legal requirements, as well as those who are fatigued or distracted, or who are not expecting to encounter a signal at a particular location. It is also necessary for traffic signals to function under a wide range of conditions including day and night, adverse weather, and visually complex surrounds.

To date, studies of traffic signal performance have not typically included observer age as an independent variable. Available evidence suggests, however, that older individuals have reduced levels of sensitivity to intensity and contrast, but not to color. Fisher (1969) reported that as a person ages, the ocular media yellows and has the effect of enhancing the contrast between a red signal and a sky background. However, this effect is more than offset by increasing light scatter within the eye, which diminishes contrast. Older drivers need increased levels of signal luminance and contrast in certain situations to perceive traffic signals as efficiently as 20- to 25-year-old drivers; however, higher signal intensities may cause disability glare. Fisher and Cole (1974), using data from Blackwell (1970), suggested that older drivers may require 1.5 times the intensity at 50 years of age and 3 times the intensity at 70 years of age, and protanopes (individuals with a color-vision deficiency resulting in partial or full insensitivity to red light) may require a fourfold increase. They noted that while increased intensity will ensure that older observers see the signal, the reaction time of older drivers will be longer than for younger drivers. To compensate for this, it would appear necessary to assume a longer required visibility distance, which would result in an increase in the signal intensity required. However, Fisher (1969) also suggested that no increase in signal intensity is likely to compensate for increasing reaction time with age. It therefore deserves emphasis that the goal of increased response times for older drivers, requiring longer visibility distances, can also be provided by ensuring that the available signal strength (peak intensity) is maintained through a wide, versus a narrow, viewing angle. This makes signal information more accessible over longer intervals.

It is generally agreed that the visibility issues associated with circular signals relate to the following factors: minimum daytime intensity, intensity distribution, size, nighttime intensity, color of signals, backplates, depreciation (light loss due to lamp wear and dirt on lenses), and phantom (apparent illumination of a signal in a facing sun). To place this discussion in context, it should also be noted that traffic signal recommendations for different sizes, colors, and in-service requirements have, in large part, been derived analytically from one research study conducted by Cole and Brown (1966).

In establishing minimum daytime intensity levels for (circular) traffic signals, the two driver characteristics that are considered with regard to the need to adjust peak intensity requirements are color anomalies and driver age. Cole and Brown (1968) determined that the *optimum* red signal intensity is 200 cd for a sky luminance of 10,000 cd/m^2, and an *adequate* signal intensity for this condition would be 100 cd. "Optimum" is defined by Cole and Brown (1968) to be the intensity that produces minimum reaction time plus 0.1 s. An "adequate" signal is one that is still not likely to be missed, although driver reaction time will be slower than for a signal of optimum intensity.

The number of organizations that specify a minimum standard for peak daytime traffic signal intensity is larger than the number of research studies upon which those standards are based. In fact, all of the standards including those for 200-mm (8-in) and 300-mm (12-in) signals, those for red, yellow, and green signals, and those for new and in-service applications are derived from a single requirement for a red traffic signal, established from the work of Cole and Brown (1966). The conclusion of this laboratory study was that a red signal with an intensity of 200 cd should invoke a certain and rapid response from an observer viewing the signal at distances up to 100 m (328 ft) even under extremely bright ambient conditions. This conclusion was based on experiments in which the background luminance was 5,142 cd/m^2. The results were linearly extrapolated to a background luminance of 10,000 cd/m^2 which yielded the 200 cd recommendation. Janoff (1990) concluded that a value of 200 cd minimum intensity for a red signal will suffice for observation distances up to 100 m (328 ft) and vehicle speeds up to 80 km/h (50 mi/h), based on analytic, laboratory, and controlled field experiments performed by Adrian (1963); Boisson and Pages (1964); Rutley, Christie, and Fisher (1965); Jainski and Schmidt-Clausen (1967); Cole and Brown (1968); Fisher (1969); and Fisher and Cole (1974). Fisher and Cole (1974) cautioned against using a value less than 200 cd, to ensure that older drivers and drivers with abnormal color vision will see the signal with certainty and with "reasonable speed."

For green signals, Fisher and Cole (1974) indicated that the ratio of green to red intensity should be 1.33:1, based on laboratory and controlled field research by Adrian (1963), Rutley et al. (1965), Jainski and Schmidt-Clausen (1967), and Fisher (1969), and the ratio of yellow to red should be 3:1, based on research performed by Rutley et al. (1965) and Jainski and Schmidt-Clausen (1967). Janoff (1990) noted that the evidence to support these ratios is somewhat variable, and support of these recommendations is mixed. Table 18, from Janoff (1990), presents the peak intensity requirements of red, green, and yellow traffic signals for 200-mm (8-in) signals for normal-speed roads and for 300-mm (12-in) signals for high-speed roads; the values presented exclude the use of backplates and ignore depreciation. A normal-speed road, in this context, includes speeds up to 80 km/h (50 mi/h), distances up to 100 m (328 ft), and sky luminances up to 10,000 cd/m^2. A high-speed road is defined as one with speeds up to 100 km/h (62 mi/h), distances up to 240 m (787 ft), and sky luminances up to 10,000 cd/m^2. Janoff also noted that although signal size is included, research performed by Cole and Brown (1968) indicated that signal size is not important because traffic signals are point sources rather than area sources and only intensity affects visibility. Thus, the required intensity can be obtained by methods other than increasing signal size (i.e., by using higher intensity sources in 200-mm signals).

Table 18. Peak (minimum) daytime intensity requirement (cd) for maintained signals with no backplate. Source: Janoff, 1990.

Signal Size	Signal Color		
	Red	Green	Yellow
200 mm (8 in)	200	265	600
300 mm (12 in)	895	1,190	2,685

The specification of standard values for *peak* intensity is important because the distribution of light intensity falls off with increasing horizontal and vertical eccentricity in the viewing angle. Janoff (1990) summarized the peak intensity standards of ITE, Commission Internationale de l'Éclairage (CIE), the British Standards Organization, and standards organizations of Australia, Japan, and South Africa. The U.S. (ITE) standard provides different recommendations for each of the three colors for each signal size. The recommendations are as follows: for red, 157 cd for 200-mm (8-in) signals and 399 cd for 300-mm (12-in) signals; for green, 314 cd for 200-mm (8-in) signals and 798 cd for 300-mm (12-in) signals; and for yellow, 726 cd for 200-mm (8-in) signals and 1,848 cd for 300-mm (12-in) signals. Australia recommends the same peak intensity for red and green (200 cd for 200-mm [8-in] signals and 600 cd for 300-mm [12-in] signals), and a yellow intensity equal to three times the red intensity. The CIE recommends the same peak intensity for all three colors (200 cd for 200-mm [8-in] signals and 600 cd for 300-mm [12-in] signals), but acknowledges that actual intensity differences between colors result due to the differential transmittance of the colored lenses (1:1.3 for red to green and 1:3 for red to yellow). Japan recommends 240 cd for all three colors. Great Britain recommends a peak intensity of 475 cd for 200-mm (8-in) red and green signals, and 800 cd for 300-mm (12-in) red and green signals. The range for red signals among all of these standards is from 157 cd (ITE) to 475 cd (British Standards Organization). The 157 cd is from research by Cole and Brown. The modal value of 200 cd, specified by Australia, South Africa, and the CIE, is based upon a depreciation factor of 33 percent.

Only two research reports provide intensity requirements for green and/or yellow signals based upon empirical data. Adrian (1963) used a subjective scale and threshold detection criteria in a study that tested red and green signals at different background luminances. He concluded that the intensity requirements for green were 1.0 and 1.2 times that of red for the subjective and threshold studies, respectively. Jainski and Schmidt-Clausen (1967) tested the ability of observers to detect the presence of a red, amber, or green spot, which was either 2 minutes or 1 degree, against varying background luminances. Their results found that green required 1.0 and 2.5 times that of red, and yellow required 2.5 and 3.0 times that of red, for 1 degree and 2 minutes, respectively. Using these results, most standards set requirements for green and yellow to be 1.3 and 3.0 times that of red, respectively. The CIE standard discusses the fact that the ratios of 1.3 and 3.0 for green and yellow appear to reflect the differences in the transmissivity of the varying color lenses.

The intensity distributions of traffic signals can be compared in two ways: percentage of peak or absolute value. For 200-mm (8-in) signals, the horizontal and vertical distributions between standards are generally similar, more so for the horizontal. The ITE distribution for 300-mm (12-in) signals is exactly the same as the ITE 200-mm (8-in) distribution. The CIE 300-mm distribution is very different from its 200-mm counterpart. The 300-mm CIE distribution requires a higher concentration of intensity in the center of the beam. The standards describe their requirements in the form of tables, which list the horizontal and vertical angular positions where a certain percentage of the peak intensity is required. Logically, maintaining a higher percentage of peak intensity for a given distance away from the center of the beam (line of sight) produces a larger angular area in which the signal indication may be perceived.

Two studies have been conducted for 200-mm (8-in) signals dealing with intensity distribution: an analytic one by Cole (1966), where the signal was in the observer's line of sight, and a controlled field experiment by Fisher (1969), where the signal was placed off the line of sight of the observer. Fisher's work is considered an extension of Cole's. Cole's work yielded an optimal distribution for traffic signals. Hulscher (1974) conducted photometric and analytic research to extend Cole's results to include 300-mm (12-in) signals. The optimum distribution for 200-mm (8-in) signals derived by Fisher (1969) is provided in figure 10, with and without a backplate. In Fisher's 1969 controlled field study, it was found that the use of a backplate permits the distribution of intensity to be more concentrated in the center of the beam. Figure 10 also presents the optimum distribution of 300-mm (12-in) signals as determined by Hulscher (1974), which was included in the 1980 CIE standard.

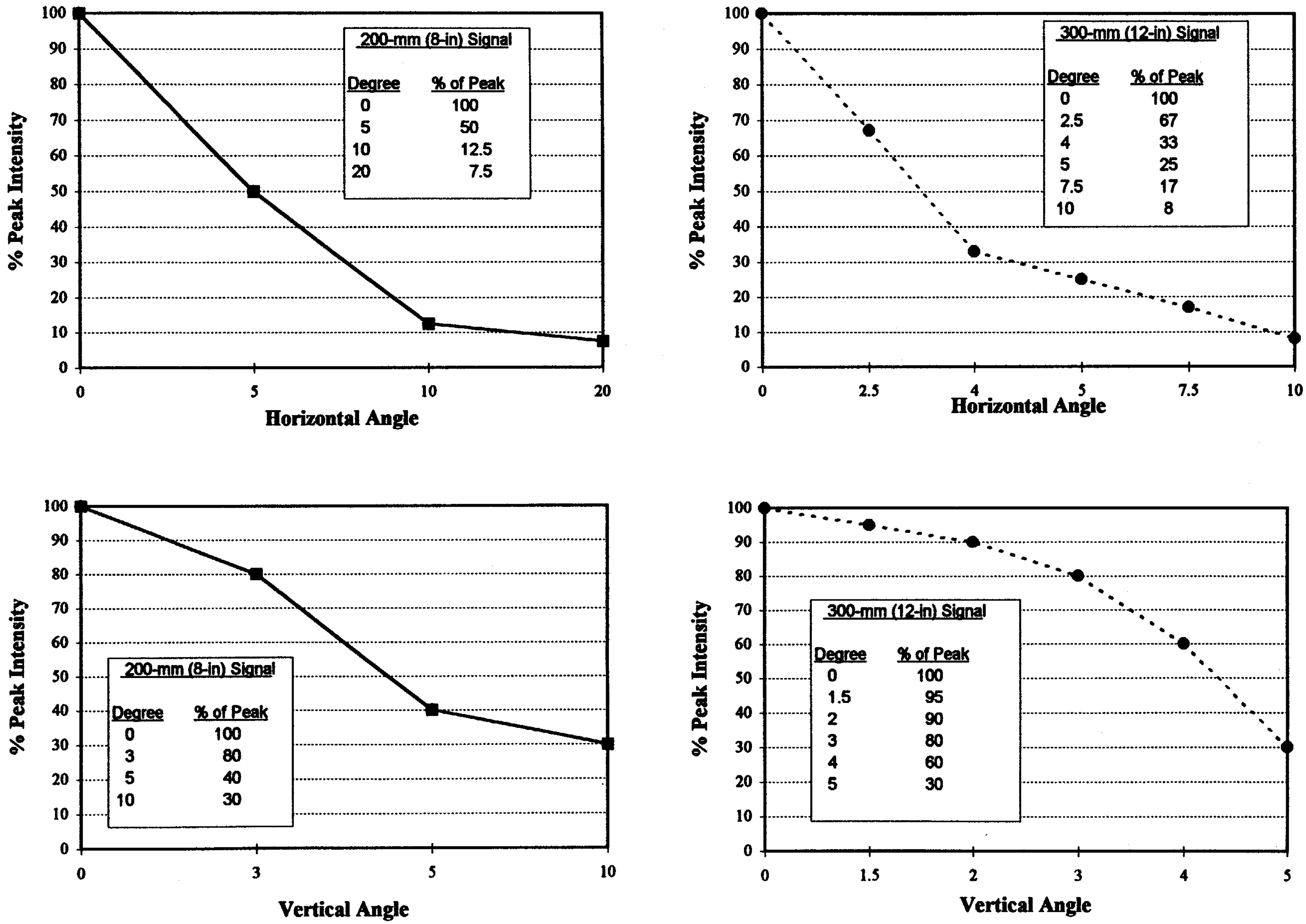


Figure 10. Optimum distribution of 200-mm and 300-mm (8-in and 12-in) traffic signals.

Regarding signal size, section 4B-8 of the MUTCD specifies the conditions for which 300-mm (12-in) signals shall be used; this section includes approaches for which the minimum visibility distance requirements (section 4B-12.1) cannot be met, approaches with 85 percentile speeds exceeding 64 km/h (40 mi/h), approaches where signalization may be unexpected, approaches with rural cross-sections where only post-mounted signals are used, and arrow (symbol) signal indications.

Some research has indicated that the dimming of signals at night may have advantages, while also reducing power consumption. Freedman, Davit, Staplin, and Breton (1985) conducted a laboratory study and controlled and observational field studies to determine the operational, safety, and economic impact of dimming traffic signals at night. Results indicated that drivers behaved safely and efficiently when signals were dimmed to as low as 30 percent of ITE recommendations. Previously, however, Lunenfeld (1977) cited the considerable range of night background luminances that may occur in concluding that in some brightly lit urban conditions, or where there is considerable visual noise, daytime signal brightness is needed to maintain an acceptable contrast ratio. The ITE standard presently does not differentiate between day and night intensity requirements. The CIE recommends that intensities greater than 200 cd or less than 25 cd be avoided at night and advises a range of 50 to 100 cd for night, except for high-speed roads where the daytime values are preferred. The South African and Australian standards allow for dimming but do not recommend an intensity level. While the option for dimming on a location-by-location basis should not be excluded, from the standpoint of older driver needs, there is no compelling reason to recommend widespread reduction of traffic signal intensity during nighttime operations.

It is common practice to try to enhance the visibility of signals by placing a large, black surround behind the signals. The backplate, rather than the sky, becomes the background of the signals, enhancing the contrast. The CIE (1988) recommends that all signals use backplates of a size (width) of three times the diameter of the signal. The ITE standard does not provide a backplate recommendation. In laboratory research, Cole and Brown (1966) found that the use of backplates reduces the required intensity by about 25 percent at distances of approximately 100 m (328 ft), where the luminance of the sky is 10,000 cd/m^2 and the speed is 80 km/h (50 mi/h), but it has little effect at longer distances unless the size of the backplate is excessive. Fisher (1969) and Fisher and Cole (1974) derived a 40-percent intensity reduction for 200-mm (8-in) signals at distances of 100 m (328 ft) and greater reductions at shorter distances, ranging up to 90 percent at distances under 25 m (82 ft). For practical purposes, a backplate three times the width of the signal reduces the intensity requirement by about 0.6 for distances up to 100 m (328 ft). In analytic computations based on Cole's work, Hulscher (1975) determined that a 300-mm (12-in) signal with a backplate requires approximately one-third less intensity.

As a practical matter, the use of a backplate would, in most cases, compensate for the effects of depreciation, since a backplate reduces the required intensity by roughly 25 percent while depreciation increases the requirement by the same amount. New guidelines published by the CIE (1988) suggest including an allowance of 25-percent transmissivity for depreciation due to dirt and aging (a 33-percent increase in intensity). The 200-cd requirement for red signals, as noted earlier, must be met *after* the depreciation factor has been taken into account.

A final issue with respect to signal performance and older drivers is the change intervals between phases, and the assumptions about perception-reaction time (PRT) on which these calculations are based. At present, a value of 1.0 s is assumed to compute change intervals for traffic signals, a value which, according to Tarawneh (1991), dates back to a 1934 Massachusetts Institute of Technology study on brake-reaction time. Tarawneh examined findings published by proponents of both "parallel" and "sequential" (serial) models of driver information processing, seeking to determine the best estimator for older individuals of a PRT encompassing six different component processing operations: (1) latency time (onset of stimulus to beginning of eye movement toward signal); (2) eye/head movement time to fixate on the signal; (3) fixation time to get enough information to identify the stimulus; (4) recognition time (interpret signal display in terms of possible courses of action); (5) decision time to select the best response in the situation; and (6) limb movement time to accomplish the appropriate steering and brake/accelerator movements.

Tarawneh's (1991) review produced several conclusions. First, the situation of a signal change at an intersection is among the most extreme, in terms of both the information-processing demand and subjective feelings of stress that will be experienced by many older drivers. Second, the most reasonable interpretation of research to date indicates that the best "mental model" to describe and predict how drivers respond in this context includes a mix of concurrent and serial-and-contingent information-processing operations. In this approach, the most valid PRT estimator will fall between the bounds of values derived from the competing models thus far, also taking into account age-related response slowing for recognition, decisionmaking, and limb movement. After a tabular summary of the specific component values upon which he based his calculations, Tarawneh (1991) called for an increase in the current PRT value used to calculate the length of the yellow interval (derived from tests of much younger subjects) from 1.0 s to 1.5 s to accommodate older drivers.

A contrasting set of results was obtained in a recent FHWA-sponsored study of traffic operations control for older drivers (Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin, 1995). This study compared the decision/response times and deceleration characteristics of older drivers (ages 60–71 and older) with those of younger drivers (younger than age 60) at the onset of the amber signal phase. Testing was conducted using a controlled field test facility, where subjects drove their own vehicles. Subjects were asked to maintain speeds of 48 km/h (30 mi/h) and 32 km/h (20 mi/h) for certain test circuits. The duration of the yellow signal was 3.0 s before turning to red. On half of the trials, the signal changed from green to yellow when the subject was 3.0 to 3.9 s from the signal, and on the remaining trials, when the subject was 4.0 to 4.9 s away from the signal. For three of the circuits, subjects were asked to brake as they normally would and to stop before reaching the intersection, if they chose to do so. During a fourth circuit, they were asked to brake to a stop, if they possibly could, if the light changed from green to yellow. Response times were measured for the drivers who stopped, from the onset of the yellow phase to the time the brake was applied.

Results of the Knoblauch et al. (1995) study showed no significant differences in 85th percentile decision/response times between younger and older drivers when subjects were close to the signal at either approach speed. The 85th percentile decision time of younger subjects was 0.39 s at 32 km/h (20 mi/h) and 0.45 s at 48 km/h (30 mi/h). For older drivers, these times were 0.51 and 0.53 s, for 32 km/h and 48 km/h (20 mi/h and 30 mi/h), respectively.

When subjects were further from the signal at amber onset, older drivers had significantly longer decision/response times (1.38 s at 32 km/h [20 mi/h] and 0.88 s at 48 km/h [30 mi/h]) than the younger drivers (0.50 s at 32 km/h [20 mi/h] and 0.46 s at 48 km/h [30 mi/h]). The authors suggested that the significant differences between older and younger drivers occurred when the subjects were relatively far from the signal, and that some older subjects will take longer to react and respond when additional time is available for them to do so. Thus, they concluded that the older drivers were not necessarily reacting inappropriately to the signal. In terms of deceleration rates, there were no significant differences, either in the mean or 15th percentile values, between the older and younger subjects. Together, these findings led the authors to conclude that no changes in amber signal phase timing are required to accommodate older drivers.

Taking the review and study findings of Tarawneh (1991) and Knoblauch et al. (1995) into consideration, an approach that retains the 1.0-s PRT value for calculating the yellow change interval but acknowledges the significant body of work documenting age-related increases in PRT, especially where there is response uncertainty, appears most reasonable. A recommendation for an all-red clearance interval logically follows, with length determined according to the ITE (1992).

O. Design Element: Fixed Lighting Installations

Table 19. Cross-references of related entries for fixed lighting installations.

Applications in Standard Reference Manuals		
MUTCD (1988)	AASHTO Green Book (1994)	Roadway Lighting Handbook Chapter 6 (1983)
Pg. 5D-1, Sect. D Pg. 2E-2, Para. 4 Pgs. 2F-7–2F-8, Sect. 2F-13	Pgs. 309-311, Sect. on *Lighting* Pg. 315, Para. 2 Pgs. 545-546, Sect. on *Lighting* Pg. 567, Para. 1 Pg. 792, Sect. on *Lighting at Intersections*	Pgs. 42-45, Sect. on *Summary of Light Sources* Pgs. 53-56, Sect. on *Classification of Luminaire Light Distributions* Pg. 96, Para(s). 2-3 Pgs. 98-99, Sect. on *Rural Intersection Lighting* Pgs. 105-106, Sect. on *Roadway Signs* and Table 7 Pgs. 120-139, Sect. on *Illumination Design Procedure*

One of the main purposes of lighting a roadway at night is to increase the visibility of the roadway and its immediate environment, thereby permitting the driver to maneuver more safely and efficiently. The visibility of an object is that property which makes it discernible from its surroundings. This property depends on a combination of factors; principally, these factors include the differences in luminance, hue, and saturation between the object and its immediate background (contrast); the angular size of the object at the eye of the observer; the luminance of the background against which it is seen; and the duration of observation.

The link between reduced visibility and highway safety, though it may be difficult to quantify in a cost benefit analysis, is conceptually straightforward. Low luminance contributes to a reduction in visual capabilities such as acuity, distance judgment, speed of seeing, color discrimination, and glare tolerance, which are already diminished capabilities in older drivers.

The Commission Internationale de l'Éclairage (1990) reports that road accidents at night are disproportionately higher in number and severity compared with accidents during the daytime. Data from 13 Organisation for Economic Co-operation and Development countries showed that the proportion of fatal nighttime accidents ranged between 25 and 59 percent (average value of 48.5 percent). In this evaluation of 62 lighting and accident studies, 85 percent of the results showed lighting to be beneficial, with approximately one-third of the results statistically significant.

In 1990, drivers (without regard to age) in the United States experienced 10.37 fatal involvements per 161 million km (100 million mi) at night and 2.25 during the day (Massie and Campbell, 1993). In their analysis, the difference between daytime and nighttime fatal rates was found to be more pronounced among younger age groups than among older ones, with drivers ages 20–24 showing a nighttime rate that was 6.1 times the daytime rate, and drivers age 75 and older showing a nighttime rate only 1.1 times the daytime rate. The lower percentage of nighttime accidents of older drivers may be due to a number of factors, including reduced exposure—older drivers as a group drive less at night—and a self-regulation process whereby those who do drive at night are the most fit and capable to perform all functional requirements of the driving task (National Highway Traffic Safety Administration, 1987).

A specific driving error with high potential for crash involvement is wrong-way movements. Analyses of wrong-way movements at intersections frequently associate these driving errors with low visibility and restricted sight distance (Vaswani, 1974, 1977; Scifres and Loutzenheiser, 1975) and note that designs that increase drivers' visibility and perception of access points to divided highways have been found to reduce the potential for accidents.

Inadequate night visibility, where the vehicle's headlights are the driver's primary light source, was reported by Vaswani (1977) as a factor that makes it more difficult for drivers to determine the correct routing at intersections with divided highways. Similarly, Woods, Rowan, and Johnson (1970) reported that locations where highway structures, land use, natural growth, or poor lighting conditions reduce the principal sources of information concerning the geometry and pavement markings are associated with higher occurrences of wrong-way maneuvers. Crowley and Seguin (1986) reported that drivers over the age of 60 are excessively involved in wrong-way movements on a per-mile basis. Suggested countermeasures include increased use of fixed lighting installations. Lighting provides a particular benefit to older drivers by increasing expectancy of needed vehicle control actions, at longer preview distances. It has been documented extensively in this *Handbook* that an older driver's ability to safely execute a *planned* action is not significantly worse than that of a younger driver. The importance of fixed lighting at intersections for older drivers can therefore be understood in terms of both the diminished visual capabilities of this group and their special needs to prepare farther in advance for unusual or unexpected aspects of intersection operations or geometry. Targets that are especially critical in this regard include shifting lane alignments; changing lane assignments (e.g., when a through lane changes to turn-only operation); a pavement width transition, particularly a reduction across the intersection; and, of course, pedestrians.

Detectability of a pedestrian is generally influenced by contrast, motion, color, and size (Robertson, Berger, and Pain, 1977). If a pedestrian is walking at night and does not have good contrast, color contrast, or size relative to other road objects, an increase in contrast will significantly improve his/her detectability. This is one effect of street lighting. Extreme contrasts as well as dark spots are reduced, giving the driver and the pedestrian a more "even" visual field. The effectiveness of fixed lighting in improving the detectability of pedestrians has been reported by Pegrum (1972); Freedman, Janoff, Koth, and McCunney (1975); Polus and Katz (1978); and Zegeer (1991).

While age-related changes in glare susceptibility and contrast threshold are currently accounted for in lighting design criteria, there are other visual effects of aging that are currently excluded from visibility criteria. Solid documentation exists of age-related declines in ocular transmittance (the total amount of light reaching the retina), particularly for the shorter wavelengths (cf. Ruddock, 1965); this suggests a potential benefit to older drivers of the "yellow tint" of high-pressure sodium highway lighting installations. The older eye experiences exaggerated intraocular scatter of light—all light, independent of wavelength (Wooten and Geri, 1987)—making these drivers more susceptible to glare. Diminished capability for visual accommodation makes it harder for older observers to focus on objects at different distances. Pupil size is reduced among older individuals through senile miosis (Owsley, 1987), which is most detrimental at night because the reduction in light entering the eye compounds the problem of light lost via the ocular media, as described above.

The loss of static and dynamic acuity—the ability to detect fine detail in stationary and moving targets—with advancing age is widely understood. Although there are pronounced individual differences in the amount of age-related reduction in static visual acuity, Owsley (1987) indicated that a loss of about 70 percent in this capability by age 85 is normal. Among other things, declines in acuity can be used to predict the distance at which text of varying size can be read on highway signs (Kline and Fuchs, 1993), under a given set of viewing conditions.

There are a number of other aspects of vision and visual attention that relate to driving. In particular, saccadic fixation, useful field of view, detection of motion in depth, and detection of angular movement have been shown to be correlated with driving performance (see Bailey and Sheedy, 1988, for a review). As a group, however, these visual functions do not appear to have strong implications for highway lighting practice, with the possible exception of the useful field of view. It could be argued that it would be advantageous to provide wider angle lighting coverage to increase the total field of view of older drivers. High-mast lighting systems can increase the field of view from 30 degrees to about 105 degrees (Hans, 1993). Such wide angles of coverage might have advantages for older drivers in terms of peripheral object detection. However, the disadvantages may outweigh the potential advantages of increasing the field of view. Although effective high-mast systems have been demonstrated (Ketvirtis and Moonah, 1995), high-mast lighting systems tend to sacrifice target contrast for increased field of view. Also, the increased visual clutter produced by the higher luminance levels in the periphery may contribute to attentional problems, particularly attention switching, that has been linked to accidents in older drivers (Summala and Mikkola, 1994). Thus, field of view is not considered as a parameter that needs to be optimized in lighting system design, especially when, with current technology, it is inevitable that visibility (i.e., target contrast) will be sacrificed.

Rockwell, Hungerford, and Balasubramanian (1976) studied the performance of drivers approaching four intersection treatments, differentiated in terms of special reflectorized delineators and signs versus illumination. A significant finding from observing 168 test approaches was that the use of roadway lighting significantly improved driving performance and earlier detection of the intersection, compared with the other treatments (e.g., signing, delineation, and new pavement markings), which showed smaller improvements in performance.

Finally, it must be emphasized that the effectiveness of intersection lighting depends upon a continuing program of monitoring and maintenance by the local authority. Guidelines published by AASHTO (1984) identify depreciation due to dirt on the luminaires and reduced lumen output from the in-service aging of lamps as factors that combine to decrease lighting system performance below design values. Maintained values in the range of 60 to 80 percent of initial design values are cited as common practice in this publication. With a particular focus on the needs of older drivers for increased illumination relative to younger motorists, to accommodate the age-related sensory deficits documented earlier in this discussion, a recommendation logically follows that lighting systems be maintained to provide service at the 80 percent level—i.e., the upper end of the practical range—with respect to their initial design values.

P. Design Element: Pedestrian Control Devices

Table 20. Cross-references of related entries for pedestrian control devices.

Applications in Standard Reference Manuals			
MUTCD (1988)	**AASHTO Green Book (1994)**	**Roadway Lighting Handbook Chapter 6 (1983)**	**NCHRP 279 Intersection Channelization Design Guide (1985)**
Pg. 2B-29 - 2B-31, Sect(s). 2B-36 & 2B-37 Pg. 3B-23, Sect. 3B-18 Pg. 3B-27, Para(s). 4-6 Pg. 3B-28, Item 2 Pgs. 4B-20–4B-21, Sect. 4B-28 Pgs. 4D-1–4D-4, Sect(s). 4D-1–4D-7	Pgs. 97-104, Sect(s). on *General Considerations, General Characteristics, Physical Characteristics, Walkway Capacities,* and *Characteristics of Persons with Disabilities* Pg. 531, Para. 3 Pg. 532, Para. 4	Pg. 18, Form 2 Pg. 30, Sect. on *Pedestrian Behavior Warrant*	Pg. 6, Table 2-2 Pg. 21, Item 9 and Fig. 3-1 Pg. 33, Bottom Right Fig. Pg. 38, Para. 1 & Top Two Fig(s). Page 39, Entire Pg.

A nationwide review of fatalities during the year 1985, and injuries during the period of 1983–1985, showed that 39 percent of all pedestrian fatalities and 9 percent of all pedestrian injuries involved the elderly (ages 64 and older) (Hauer, 1988). While the number of injuries is close to the population distribution (approximately 12 percent), the number of fatalities far exceeds the proportion of older pedestrians. The percentages of pedestrian fatalities and injuries occurring at intersections were 33 percent and 51 percent, respectively (Hauer, 1988). Accident types that predominantly involve older pedestrians at intersections are as follows (Blomberg and Edwards, 1990):

- Vehicle turn/merge—The vehicle turns left or right and strikes the pedestrian.
- Intersection dash—A pedestrian appears suddenly in the street in front of an oncoming vehicle at an intersection.
- Multiple threat—One or more vehicles stop in the through lane, usually at a crosswalk at an unsignalized intersection. The pedestrian steps in front of the stopped vehicle(s) and into the path of a through vehicle in the adjacent lane.
- Bus-stop related—The pedestrian steps out from in front of a stopped bus and is struck by a vehicle moving in the same direction as the bus.
- Pedestrian trapped—At a signalized intersection, a pedestrian is hit when a traffic signal turns red (for the pedestrian) and cross-traffic vehicles start moving.
- Nighttime—A pedestrian is struck at night when crossing at an intersection.

Earlier analyses of over 5,300 pedestrian accidents occurring at urban intersections indicated that a significantly greater proportion of pedestrians age 65 and older were hit at signalized intersections than any other group (Robertson, Berger, and Pain, 1977).

Age-related diminished capabilities, which may make it more difficult for older pedestrians to negotiate intersections, include decreased contrast sensitivity and visual acuity, reduced peripheral vision and useful field of view, decreased ability to judge safe gaps, slowed walking speed, and physical limitations resulting from arthritis and other health problems. Older pedestrian problem behaviors include a greater likelihood to delay before crossing, to spend more time at the curb, to take longer to cross the road, and to make more head movements before and during crossing (Wilson and Grayson, 1980).

Older and Grayson (1972) reported that although older pedestrians involved in accidents looked more often than the middle-aged group studied, over 70 percent of the adults struck by a vehicle reported not seeing it before impact. In a survey of older pedestrians (average age of 75) involved in accidents, 63 percent reported that they failed to see the vehicle that hit them, or to see it in time to take evasive action (Sheppard and Pattinson, 1986). Knoblauch, Nitzburg, Dewar, Templer, and Pietrucha (1995) noted that difficulty seeing a vehicle against a (complex) street background may occur with vehicles of certain colors, causing them to blend in with their background. This is especially problematic for older persons with reduced contrast sensitivity, who require a higher contrast for detection of the same targets than younger individuals, and who also have greater difficulty dividing attention between multiple sources and selectively attending to the most relevant targets. In addition, the loss of peripheral vision increases an older pedestrian's chances of not detecting approaching and turning vehicles from the side.

Reductions in visual acuity make it more difficult for older pedestrians to read the crossing signal. In a survey of older pedestrians in the Orlando, Florida, area, 25 percent of the participants reported difficulty seeing the crosswalk signal from the opposite side of the street (Bailey, Jones, Stout, Bailey, Kass, and Morgan, 1992).

Older pedestrians wait for longer gaps between vehicles before attempting to cross the road. In one study, approximately 85 percent of the pedestrians age 60 and older required a minimum gap of 9 s before crossing the road, while only 63 percent of all pedestrians required this minimum duration (Tobey, Shungman, and Knoblauch, 1983). The decline in depth perception may contribute to older persons' reduced ability to judge gaps in oncoming traffic. It may be concluded from these studies that older pedestrians do not process information (presence, speed, and distance of other vehicles) as efficiently as younger pedestrians, and therefore require more time to reach a decision. Other researchers have observed that older pedestrians do not plan their traffic behavior, are too trusting about traffic rules, fail to check for oncoming traffic before crossing at intersections, underestimate the speed of approaching vehicles, and follow other pedestrians without first checking for conflicts before crossing (Jonah and Engel, 1983; Mathey, 1983).

With increasing age, there is a concurrent loss of physical strength, joint flexibility, agility, balance, coordination and motor skills, and stamina. These losses contribute to slower walking speeds and difficulty negotiating curbs. In addition, older persons often fall as a result of undetected surface irregularities in the pavement and misestimation of curb heights. This results from a decline in contrast sensitivity and depth perception. In an assessment of 81 older residents (ages 70–97) to examine susceptibility to falling, 58 percent experienced a fall in the year following clinical assessment (Clark, Lord, and Webster, 1993). Impaired cognition, abnormal reaction to any push or pressure, history of palpitations, and abnormal stepping were each associated with falling. Knoblauch, Nitzburg, Reinfurt, Council, Zegeer, and Popkin (1995) reported that locating the curb accurately and placing the foot is a matter of some care, particularly for the elderly, the very young, and those with physical disabilities.

The studies discussed below define the types of accidents in which older pedestrians are most likely to be involved, and under what conditions the accidents most frequently occur. In addition, the specific geometric characteristics, traffic control devices (including signs, signals, and markings), and pedestrian signals that seem to contribute to older pedestrians' difficulties at intersections are discussed.

Zegeer and Zegeer (1988) stressed the importance of "tailoring" the most appropriate traffic control measures to suit the conditions at a given site. The effect of any traffic control measure is highly dependent on specific locational characteristics, such as traffic conditions (e.g., volumes, speeds, turning movements), pedestrian volumes and pedestrian mix (e.g., young children, college students, older adults, persons with physical disabilities), street width, existing traffic controls, area type (e.g., rural, urban, suburban), site distance, accident patterns, presence of enforcement, and numerous other factors.

Harrell (1990) used distance stood from the curb as a measure of pedestrian risk for intersection crossing. Observations of 696 pedestrians divided among 3 age groups (age 30 and under, ages 31–50, and age 51 and older) showed that the oldest group stood the farthest from the curb, that they stood even farther back under nighttime conditions, and that older females stood the farthest distance from the curb. The author used these data to dispel the findings in the literature that older pedestrians are not cognizant of the risks of exposure to injury from passing vehicles. Similarly, it may be argued that this behavior keeps them from detecting potential conflict vehicles and makes speed and distance judgments more difficult for them, while limiting their conspicuity to approaching drivers who might otherwise slow down if pedestrians were detected standing at the curbside at a crosswalk.

A study of pedestrian accidents conducted at 31 high-pedestrian accident sections in Maryland between 1974 and 1976 showed that pedestrians age 60 and older were involved in 53 (9.6 percent) of the accidents, and children younger than age 12 showed the same proportions. The pedestrians age 60 and older accounted for 25.6 percent of the fatal accidents. Compliance with traffic control devices was found to be poor for all pedestrians at all study locations; it was also found that most pedestrians keyed on the moving vehicle rather than on the traffic and pedestrian control devices. Only when the traffic volumes were so high that it was impossible to cross did pedestrians rely on traffic control devices (Bush, 1986).

Garber and Srinivasan (1991) conducted a study of 2,550 accidents involving pedestrians that occurred in the rural and urban areas of Virginia to identify intersection geometric characteristics and intersection traffic control devices that were predominant in crashes involving older pedestrians. Accident frequency by location and age for the accidents within the cities showed that while the highest percentage of accidents involving pedestrians age 59 and younger occurred within 46 m (150 ft) from the intersection stop line, the highest percentage of accidents for pedestrians age 60 years and older (51.8 percent) occurred *within the intersection*.

More recently, Knoblauch, Nitzburg, Reinfurt, et al. (1995) reported that, compared with younger pedestrians, older adults are overinvolved in crashes while crossing streets at intersections. In their earlier analysis of the national Fatal Accident Reporting System (FARS) data for the period 1980–1989, 32.2 and 35.3 percent of the deaths for pedestrians ages 65–74 and age 75 and older, respectively, occurred at intersections (Reinfurt, Council, Zegeer, and Popkin, 1992). This compared with 22 percent or less for the younger age groups. Analysis of the North Carolina motor vehicle crash file for 1980–1990 displayed somewhat smaller percentages, but showed the trend of increasing pedestrian accidents at intersections as age increased.

Further analysis of the North Carolina database showed that pedestrians age 65 and older as well as those ages 45–64 experienced 37 percent of their accidents on roadways with four or more lanes. This compares with 23.7 percent for pedestrians ages 10–44 and 13.6 percent for those age 9 and younger. The highest number of pedestrian-vehicle crashes occurred when the vehicle was going straight (59.7 percent), followed by a vehicle turning left (17.2 percent), and a vehicle turning right (13.3 percent). Right-turn crashes accounted for 18.9 percent of crashes with pedestrians ages 65–74, compared with 14.2 percent for pedestrians age 75 and older. The oldest pedestrian group was the most likely to be struck by a left-turning vehicle; they accounted for 23.9 percent of the crashes, compared with 18.1 percent of those ages 65–74 and 15.8 percent of those ages 45–64.

Knoblauch, Nitzburg, Reinfurt, et al. (1995) conducted a study to determine if pedestrian comprehension of and compliance with pedestrian signals could be improved by installing a placard that explained the three phases of pedestrian signals. They used findings from: (1) a focus group and workshop conducted in Baltimore, Maryland, with 13 participants ages 19–62 and (2) questionnaires administered to 225 individuals ages 19–80 and older at four Virginia Department of Motor Vehicles offices to determine the most effective message content and format for a pedestrian signal education placard. The newly developed placard was installed at six intersections in Virginia, Maryland, and New York. Observational studies of more than 4,300 pedestrians during 600 signal cycles found no change in pedestrian signal compliance. However, results from questionnaires administered to 92 subjects at Departments of Motor Vehicles in Virginia, Maryland, and New York indicated a significant increase in understanding of the phases of the pedestrian signal. The authors concluded that although pedestrian crossing behavior is more influenced by the presence or absence of traffic than the signal indication, the wording on the placard was based on quantitative procedures using a relatively large number of subjects and should be used where signal educational placards are installed. The wording of the educational placard recommended by Knoblauch, Nitzburg, Reinfurt, et al. (1995) is shown in Recommendation 2 of Design Element P. A modification for a two-stage crossing is shown in Recommendation 3.

Zegeer and Cynecki (1986) tested a LOOK FOR TURNING VEHICLES pavement marking in a crosswalk, as a low-cost countermeasure to remind pedestrians to be alert for turning vehicles, including right-turn-on-red (RTOR) vehicles. Results showed an overall reduction in conflicts and interactions for RTOR vehicles and also for the total number of turning vehicles. Even with an RTOR prohibition, approximately 20 percent of motorists committed an RTOR violation when given the opportunity (Zegeer and Cynecki, 1986). Of those violations, about 23.4 percent resulted in conflicts with pedestrians or vehicles on the side street.

Zegeer, Opiela, and Cynecki (1982) conducted an accident analysis to determine whether pedestrian accidents are significantly affected by the presence of pedestrian signals and by different signal timing strategies. They found no significant differences in pedestrian accidents between intersections that had standard-timed (concurrent walk) pedestrian signals compared with intersections that had no pedestrian signals. Concurrent or standard timing provides for pedestrians to walk concurrently (parallel) with traffic flow on the WALK signal. Vehicles are generally permitted to turn right (or left) on a green light while pedestrians are crossing on the WALK interval. Other timing strategies include early release timing, late release timing, and exclusive timing. In early release timing, the pedestrian WALK indication is given before the parallel traffic is given a green light, allowing pedestrians to get a head start into the crosswalk before vehicles are permitted to turn. In late release timing, the pedestrians are held until a portion of the parallel traffic has turned. Exclusive timing is a countermeasure where traffic signals are used to stop motor vehicle traffic in all directions simultaneously for a phase each cycle, while pedestrians are allowed to cross the street. "Barnes Dance" or "scramble" timing is a type of exclusive timing where pedestrians may also cross diagonally in addition to crossing the street. Exclusive timing is intended to virtually eliminate turning traffic or other movements that conflict with pedestrians while they cross the street. In the Zegeer et al. (1982) analysis, exclusive-timed locations were associated with a 50 percent decrease in pedestrian accidents for intersections with moderate to high pedestrian volumes when compared with both standard-timed intersections and intersections that had no pedestrian signals. However, this timing strategy causes excessive delays to both motorists and pedestrians. Older road users (age 65 and older) recommended the following pedestrian-related countermeasures for pedestrian signs and signals, during focus group sessions held as a part of the research conducted by Knoblauch, Nitzburg, Reinfurt, et al. (1995): (1) reevaluate the length of pedestrian walk signals due to increasingly wider highways, (2) implement more Barnes Dance signals at major intersections, and (3) provide more YIELD TO PEDESTRIANS signs in the vicinity of heavy pedestrian traffic.

The MUTCD (1988) indicates that a pedestrian clearance interval shall be provided when pedestrian signal indications are used, and should consist of a flashing DON'T WALK interval of sufficient duration to allow a pedestrian crossing in the crosswalk to leave the curb and travel to the center of the farthest traveled lane before opposing vehicles receive a green indication. The MUTCD (1988) assumes a normal walking speed of 1.22 m/s (4.0 ft/s). The *Transportation and Traffic Engineering Handbook* (Institute of Transportation Engineers [ITE], 1982), states that for relatively slow walkers, speeds of from 0.91 to 0.99 m/s (3.0 to 3.25 ft/s) would be more appropriate. Older pedestrian walking speed has been studied by numerous researchers. Sleight (1972) determined that there would be safety justification for use of speeds between 0.91 and 0.99 m/s (3.0 to 3.25 ft/s), based on the results of a study by Sjostedt (1967). In this study, average adults and the elderly had walking speeds of 1.37 m/s (4.5 ft/s); however, 20 percent of the older pedestrians crossed at speeds slower than 1.22 m/s (4.0 ft/s). The 85th percentile older pedestrian walking speed in that study was 1.04 m/s (3.4 ft/s). A 1982 study by the Minnesota Department of Transportation found that the average walking speed of older pedestrians was 0.91 m/s (3.0 ft/s). In a study conducted in Florida, it was found that a walking speed of 0.76 m/s (2.5 ft/s) would accommodate 87 percent of the older pedestrians observed (ITE, undated). Weiner (1968) found an average rate for all individuals of 1.29 m/s (4.22 ft/s), and of 1.13 m/s (3.7 ft/s) for women only. A Swedish study by Dahlstedt (undated), using pedestrians age 70 and older, found that the 85th percentile comfortable crossing speed was 0.67 m/s (2.2 ft/s).

Hoxie and Rubenstein (1994) measured the crossing times of older and younger pedestrians at a 21.85-m- (71.69-ft-) wide intersection in Los Angeles, CA, and found that older pedestrians (age 65 and older) took significantly longer than younger pedestrians to cross the street. In this study, the *average* walking speed of the older pedestrians was 0.86 m/s (2.8 ft/s), with a standard deviation of 0.17 m/s (0.56 ft/s); the average speed of the younger pedestrians was 1.27 m/s (4.2 ft/s), with a standard deviation of 0.17 m/s (0.56 ft/s). Of the 592 older pedestrians observed, 27 percent were unable to reach the curb before the light changed to allow cross traffic to enter the intersection, and one-fourth of this group were stranded at least a full traffic lane away from safety.

More recently, Knoblauch, Nitzburg, Dewar, et al. (1995) conducted a series of field studies to quantify the walking speed, start-up time, and stride length of pedestrians younger than age 65 and pedestrians 65 and older under varying environmental conditions. Analysis of the walking speeds of 3,458 pedestrians younger than age 65 and 3,665 pedestrians age 65 and older crossing at intersections showed that the mean walking speed for younger pedestrians was 1.51 m/s (4.95 ft/s) and for older pedestrians was 1.25 m/s (4.11 ft/s). The 15th percentile speeds were 1.25 m/s and 0.97 m/s (4.09 ft/s and 3.19 ft/s) for younger and older pedestrians, respectively. These differences were statistically significant. Among the many additional findings with regard to walking speed were the following: pedestrians who start on the WALK signal walk slower than those who cross on either the flashing DON'T WALK or steady DON'T WALK; the slowest walking speeds were found on local streets while the faster walking speeds were found on collector-distributors; sites with symbolic pedestrian signals had slower speeds than sites with word messages; pedestrians walk faster where RTOR is not permitted, where there is a median, and where there are curb cuts; faster crossing speeds were found at sites with moderate traffic volumes than at sites with low or high vehicle volumes.

For design purposes, a separate analysis was conducted by Knoblauch, Nitzburg, Dewar, et al. (1995) for pedestrians who complied with the signal, as they tended to walk more slowly than those who crossed illegally. The mean crossing speed for the young compliers was 1.46 m/s (4.79 ft/s) and for the older compliers was 1.20 m/s (3.94 ft/s). The 15th percentile speed for the young compliers was 1.21 m/s (3.97 ft/s) and was 0.94 m/s (3.08 ft/s) for the older compliers. Older female compliers showed the slowest walking speeds, with a mean speed of 1.14 m/s (3.74 ft/s) and a 15th percentile of 0.91 m/s (2.97 ft/s). One of the slowest 15th percentile values (0.89 m/s [2.94 ft/s]) was observed for older pedestrians crossing snow-covered roadways. It was concluded from this research that a mean design speed of 1.22 m/s (4.0 ft/s) is appropriate, and where a 15th percentile is appropriate, a walking speed of 0.91 m/s (3.0 ft/s) is reasonable. It was also determined by Knoblauch, Nitzburg, Dewar, et al. (1995) that the slower walking speed of older pedestrians is due largely to their shorter stride lengths. The stride lengths of all older pedestrians are approximately 86 percent of those of younger pedestrians.

Finally, Knoblauch, Nitzburg, Dewar, et al. (1995) also measured start-up times for younger and older pedestrians who stopped at the curb and waited for the signal to change before starting to cross. The mean value for younger pedestrians was 1.93 s compared with 2.48 s for older pedestrians. The 85th percentile value of 3.06 s was obtained for younger pedestrians, compared with 3.76 s for older pedestrians. For design purposes, the authors concluded that a mean value of 2.5 s and an 85th percentile value of 3.75 s would be appropriate. These data specifically did not include pedestrians using a tripod cane, a walker, or two canes; people in wheelchairs; or people walking bikes or dogs. The MUTCD (1988) states that under normal conditions, the WALK interval should be at least 4 to 7 s in length so that pedestrians will have adequate opportunity to leave the curb before the clearance interval is shown. Parsonson (1992) noted that the reason this much time is needed is because many pedestrians waiting at the curb watch the traffic, and not the signals. When they see conflicting traffic coming to a stop, they will then look at the signal to check that it has changed in their favor. If they are waiting at a right-hand curb, they will often take time to glance to their left rear to see if an entering vehicle is about to make a right turn across their path. Parsonson reported that a pedestrian reasonably close to the curb and alert to a normal degree can be observed to require up to 4 or 5 s for this reaction, timed from when the signal changes to indicate that it is safe to cross, to stepping off the curb. It may be remembered that older pedestrians stand farther away from the curb, and may or may not be alert. In addition, there are many drivers who run the amber and red signals, and it is prudent for pedestrians to "double-check" that traffic has indeed obeyed the traffic signal, and that there are no vehicles turning right on red or (permissive) left on green before proceeding into the crosswalk. Because older persons have difficulty dividing attention, this scanning and decisionmaking process requires more time than it would for a younger pedestrian. Parsonson (1992) reported that the State of Delaware has found that pedestrians do not react well to the short WALK and long flashing DON'T WALK timing pattern. They equate the flashing with a vehicle yellow period. The Florida Department of Transportation and the city of Durham, Ontario, provide sufficient WALK time for the pedestrian to reach the middle of the street, so that the pedestrian will not turn around when the flashing DON'T WALK begins.

II. INTERCHANGES (GRADE SEPARATION)

The following discussion presents the rationale and supporting evidence for *Handbook* recommendations pertaining to these four design elements (A–D):

A. Exit Signing and Exit Ramp Gore Delineation
B. Acceleration/Deceleration Lane Design Features
C. Fixed Lighting Installations
D. Traffic Control Devices for Prohibited Movements on Freeway Ramps

A. Design Element: Exit Signing and Exit Ramp Gore Delineation

Table 21. Cross-references of related entries for exit signing and exit ramp gore delineation.

Applications in Standard Reference Manuals	
MUTCD (1988)	**AASHTO Green Book (1994)**
Pg. 2B-19, Sect(s). 2B-26 & 2B-27 Pg. 2B-20, Para(s). 1-2 Pg. 2D-3, Sect. 2D-8 Pg. 23D-4, Figure 2-6 Pg. 2E-2, Sect. 2E-4 Pg. 2E-3, Para(s) 3-4 & 5 Pgs. 2E-5–2E-6, Items G, I, M, & O of Table II-1 Pg. 2E-7, Sect(s). 2E-15 & 2E-16 Pgs. 2E-10–2E-16, Sect(s). 2E-24, & 2E-26 - 2E-30 Pg. 2E-18, Item 1 Pg. 2E-21, Sect. 2E-36 Pgs. 2E-22 & 2E-33, Sect. 2E-40 Pgs. 2F-5–2F-7, Items A, B, D, G, I, & M-O of Table II-2 Pg. 2F-8, Sect(s). 2f-14 & 2F-15 Pgs. 2F-9–2F-34, Sect(s). 2F-19 - 2F-20 & 2F-23 - 2F-32 Pgs. 3B-14–3B-16, Sect. 3B-11 Pg. 3D-1, Para(s) 3-4 Pg. 3D-2, Para(s) 4 & 8 Pg. 3D-3, Sect. 3D-5	Pg. 897, Para. 5 Pg. 276, Item 8 Pg. 927, Para(s). 1, 3 Pg. 929, Para. 6 Pgs. 933-934, Figures X-68 & X-70

A motorist's ability to use highway information from signing and delineation is governed by *information acquisition*, or how well the source can be seen. It is also governed by *information processing*, or the speed and accuracy with which the message content can be understood. When either of these key aspects of driver performance is compromised, the result is delayed decisionmaking, erratic behavior, and maneuver errors.

Taylor and McGee (1973) investigated driver behavior at exit gore areas to determine the causes and characteristics of erratic maneuvers. Interviews were also conducted with many drivers whose actions at the gore area were indicative of route choice difficulties. Analyses of the patterns of erratic maneuvers (cross gore paint, cross gore area, stop in gore, back up, sudden slowing, lane change, swerve, stop on shoulders) and on-site driver interviews were used to determine causative factors of these maneuvers. The most frequent erratic maneuver was crossing the gore paint, which had a 69 percent relative frequency of occurrence for drivers exiting, and a 61 percent relative frequency of occurrence for drivers traveling through the interchange. Most of the motorists who made erratic maneuvers (77 percent) were unfamiliar with the route on which they were traveling. Interviews with exiting motorists who made erratic maneuvers indicated that more than half of the drivers were not adequately prepared for the exit. These drivers indicated that the signs lacked needed information or that the information was misleading. Interviews, with drivers who made erratic maneuvers and continued through, indicated that approximately one-half had difficulty identifying their direction. Approximately 35 percent stated the signing was not clear, 21 percent responded they could not clearly distinguish the location of the exit ramp, and 34 percent thought the road markings were inadequate.

The following discussion of exit signing issues focuses on the legibility of text, the understandability of diagrammatic guide signs, and the placement of devices to provide needed message redundancy while avoiding information overload.

Current sign legibility standards assume that a 25-mm (1-in) tall letter is legible at 15.2 m (50 ft), which roughly corresponds to a visual acuity of 20/25; as documented in the Transportation Research Board's *Special Report 218* (1988), this "legibility index" value of 50 ft/in exceeds the visual ability of 30 to 40 percent of drivers who are 65–74 years of age, even under favorable contrast conditions. A 0.48 m/mm (40 ft/in) standard can generally be achieved by older drivers for signs with contrast ratios (between the legend and background) greater than 5:1 (slightly higher for guide signs) and luminance greater than 10 cd/m^2 (candelas per square meter) for partially reflectorized signs. A more conservative standard, which corresponds to 20/40 vision, would be a legibility index of 0.36 m/mm (30 ft/in).

Nighttime legibility requirements were addressed by Staplin, Lococo, and Sim (1990), who conducted a laboratory simulation using 28 young/middle-aged subjects (ages 19–49) and 30 older subjects (ages 65–80) to measure age-related differences in drivers' ability to read unique word combinations (of four letters) on green-and-white guide signs. As expected, older drivers required significantly larger letter sizes to read the (unfamiliar) words than younger drivers. Translating the 6-m (20-ft) subject-to-stimulus distance in the laboratory to a requirement of 183 m (600 ft) to read a freeway sign, the data showed that older subjects would require a letter height of 600 mm (24 in), corresponding to an acuity of 20/46. This corresponds to a legibility index of 0.3 m/mm (25 ft/in), for positive contrast (lighter characters on darker background) highway guide signs.

In a review of State practices, McGee (1991) reported that Oregon reduced the size of letters on their freeway signs from 333 mm (13.33 in) uppercase and 250 mm (10 in) lowercase to 200 mm (8 in) and 150 mm (6 in), respectively. They received numerous complaints that the signs were difficult to read at highway speeds and they therefore returned the letter sizes to their original heights (George, 1987). By contrast, North Carolina, in consideration of older driver needs, increased the Interstate shield size from 900 to 1200 mm (36 in to 48 in), the uppercase letter size from 400 mm (16 in) to 500 mm (20 in), and the lowercase letter size from 300 mm (12 in) to 375 mm (15 in) on guide signs at freeway-to-freeway interchanges (McGee, 1991).

As suggested by the preceding discussion, motorists' responses to highway sign information also depend upon its ease of recall, which in turn is related to reading time. Reading time is the time it actually takes a driver to read a sign message, contrasted with exposure time or available viewing time, which is the length of time a driver is within the legibility distance of the message. As drivers travel, they must look away from the highway to read signs posted overhead or at the side of the road, and then back to the roadway. During each glance, the maximum amount of text that can be read is three to four *familiar* words or abbreviations. A motorist's rapid understanding and integration of message components in memory will greatly assist his/her recall of the message while deciding upon a response. Two errors in message presentation must be avoided: (1) providing too much information in too short a time and (2) providing ambiguous information that leaves either the intent of the message or the desired driver response uncertain.

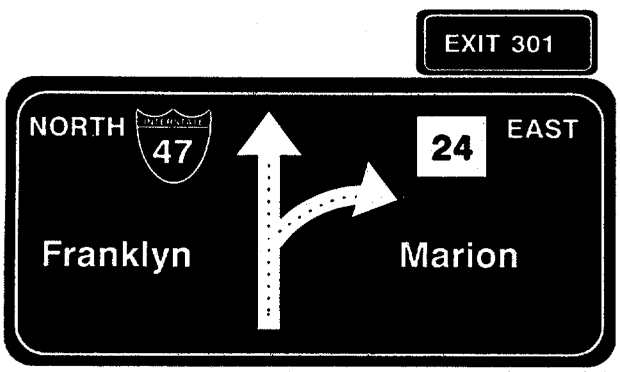
Mace, Hostetter, and Seguin (1967) conducted a laboratory, controlled field, and observational field study to evaluate how information presentation time (the amount of time that a sign is readable to a driver) and information lead distance (the distance from an exit that the advance sign is placed) affect exiting behavior at freeway interchanges. They found that 403 m (0.25 mi) is inadequate for information lead distance and, because there were few differences in driver exiting behavior with information lead distances of 805 m (0.5 mi) and 1,610 m (1.0 mi), that 805 m (0.5 mi) is optimal. In addition, a viewing time of 5 s was adequate for signs containing one to four pieces of information. Lunenfeld (1993) noted that a driver's short-term memory span is between 0.5 and 2 min, and that drivers may forget advance interchange information messages if the time span between the advance notification and the exit ramp exceeds the memory limit. He advocates the use of repetition for interchange information treatments (multiple/successive signs), which will also aid in situations where a sign is blocked by foliage or trucks. The *MUTCD* (section 2E-26) states that "*for major and intermediate interchanges, two and preferably three advance guide signs should be used. The recommended location for their placement is one-half, one, and two miles in advance of the exit. However, where this is not practicable, the distance shown should be to the nearest 1/4 mi.*" It further states that "*at minor interchanges, only one advance guide sign is required. It should be located 1/4 to 1/2 mi from the exit gore.*" In light of the age-related diminished capabilities discussed in this and related *Handbook* sections, an extension of the recommendation for major and intermediate interchanges to minor interchanges appears justified.

The effect of diagrammatic signing on driver performance at freeway interchanges was studied by numerous researchers in the early 1970's. Bergen (1970) found that graphic guide signs permitted significantly better route guidance performance than conventional signs on certain interchanges, such as collector-distributor with lane drop and multiple split ramps. In pilot studies conducted in New Jersey, Roberts (1972) found that diagrammatic signs that included lane lines were more effective (resulted in a significant reduction in erratic maneuvers) than conventional signs at the interchange of I-287 and U.S. 22, a complex interchange with both left- and right-side exits. Flener (1972) commented on the difficulty in evaluating the effectiveness of traffic control devices in reducing erratic maneuvers at exit gore areas using before and after designs, due to the "novelty effect." Although Roberts (1972) noted that the change *could* be attributed to the greater attention-getting value of novel signs, it was demonstrated that diagrammatic guide signs provide advance information that is readable at a farther distance than that provided by conventional sign text, as well as information about the number of lanes available for any one movement.

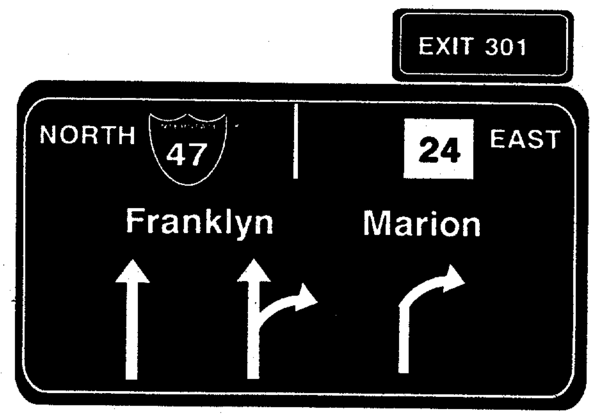
Roberts, Reilly, and Jagannath (1974) studied the effectiveness of diagrammatic versus conventional guide signs in a field study at 10 sites. The results were mixed. Several sites showed a reduction in stopping, backing, or weaving erratic maneuvers after installation of the diagrammatic signs. Some sites showed a reduction in stopping and backing maneuvers but an increase in weaving maneuvers (or vice versa). Still other sites showed no change as a function of sign type. Stopping and backing erratic maneuvers were reduced, however, at 9 of the 10 sites.

Taylor and McGee (1973) noted that the main advantage of diagrammatic signing lies in the ability to provide information regarding the interchange layout prior to the exit area. Sign format, however, remains an issue. Conflicting evidence on the effectiveness of diagrammatic signs was reported by Gordon (1972), who found that conventional signs produced fewer lane-placement errors and errors on exit lanes and were more quickly responded to than experimental diagrammatic signs tested at six interchanges in a laboratory study. At the same time, an analysis of particular diagrammatic designs showed that when a diagrammatic sign provided a single arrow or a forked arrow, reaction time was faster and there were fewer errors compared with the conventional sign. Zajkowski and Nees (1976) studied subject response time and correctness of lane choice as a function of sign type, in the laboratory. They found that response times were consistently longer for diagrammatic signs than for conventional signs; however, the difference may have been attributable to an increase in information on diagrammatic signs. There were more correct lane-choice responses for conventional signs, and subjects reported more confidence in their lane-choice decisions and a preference for conventional signs. Mast, Chernisky, and Hooper (1972) found that some drivers may require more time to read and interpret information on diagrammatic signs in comparison with conventional signs, and driver information interpretation time may increase as the graphic component of the sign becomes more complex.

More recently, Brackett, Huchingson, Trout, and Womack (1992) conducted a survey of 662 drivers in 3 age groups (younger than age 25, ages 25-54, and 55 and older) comparing alternative methods of providing lane assignment information on freeway guide signs. The findings of several comparisons in the research are reported, although no analyses using age as an independent variable were performed. First, when two common routes were displayed side by side on an exit guide sign, approximately one-half of the drivers believed that the destinations referred to different routes to be accessed by different lanes (i.e., drivers spatially cluster information with each arrow, assuming that information located on the left side of a sign is associated with an arrow also on the left side, and information on the right side is associated with EXIT ONLY or EXIT ONLY with an arrow). When destinations were arrayed one below another, 85 percent of the drivers understood that they were a common route. Second, white downward arrows used in a side-by-side format with an EXIT ONLY (E11-1) panel to indicate that two lanes could exit, were misunderstood by 80 percent of the subjects. Third, 56 percent of drivers misinterpreted the phrase NEXT RIGHT on conventional signs as an indication of a mandatory exit, and 30 percent misinterpreted the phrase NEXT LEFT in the same manner, when these signs were placed over the right and left lanes, respectively. Fourth, when conventional MUTCD diagrammatic signs were compared with modified diagrammatic signs which provided separate arrows for each lane, the modified diagrammatic signs resulted in a 13–17 percent greater understanding of when a lane must exit and when an adjacent lane may exit or continue through (two-lane exit with optional lane). When the number of arrow shafts exceeded the number of lanes (for example, when there is an added right-hand lane downstream of the overhead sign), less than 30 percent of the respondents understood that there would be an added exit lane upstream on the right. With one arrow per lane, comprehension increased by 28 percent over when there were more arrows than lanes (optional use or added lanes). Figure 11 displays: (a) an example of a conventional diagrammatic sign (from the MUTCD figure 2-30) and (b) a modified diagrammatic for this exit situation.



(a)



(b)

Figure 11. Example of signing used by Brackett, Huchingson, Trout, and Womack (1992) to compare (a) comprehension of MUTCD diagrammatics and (b) modified diagrammatics.

The following discussion of exit ramp gore delineation focuses on studies of which treatments are necessary to ensure rapid and accurate detection of the gore location and ramp heading, particularly under nighttime or reduced visibility conditions.

Taylor and McGee (1973) reported that the location of the gore is usually perceived easily during daylight hours, because a driver can rely on a direct view of the geometry, as well as signing and delineation. However, this task becomes considerably more difficult during darkness, because the driver can no longer rely on a direct view of the geometry; and exit gore signing may be misleading because of the inconsistency in the distance at which it is placed from the nose of the gore area from location to location. At night, delineation is probably the most beneficial information source to the exiting motorist, because it outlines and therefore pinpoints the location of the gore.

Taylor and McGee (1973) measured the effects of the presence of gore area delineation on driver performance at night, to determine which of various delineation devices (pavement markings, post delineators, raised pavement markers (RPMs), and a combination of treatments) were most effective. Measures of effectiveness included the point of entry into the deceleration lane, the exiting speed, and any erratic maneuvers. Two right-hand exits, one with a parallel-lane type of deceleration lane and one with a direct-taper type, were selected as test sites. Specifically, the treatment conditions were: (1) post delineator treatment—amber post delineators placed along the ramp edge of the gore area, plus crystal delineators positioned along the through side; (2) RPM treatment—amber RPMs placed on the ramp side of the gore paint markings, plus crystal RPMs on the through side; and (3) combination treatment—the post delineator treatment and the RPM treatment installed in combination.

The baseline condition for this study was moderately worn painted diagonal gore markings and edgelines, with no other delineation devices. All three delineation treatments produced earlier points of entry into the deceleration lane than under the baseline condition. The RPMs were more effective than the post delineators and produced earlier exiting points. The earliest exiting points were found with the combination of RPMs and post delineators. Gore area delineation reduced the frequency of erratic maneuvers at night at both sites. The RPM technique and combination treatment produced significantly lower exiting speeds than did the use of post delineators at one site, and all three treatments produced lower exiting speeds compared with the baseline condition.

Other researchers have also evaluated the effects of RPMs at exit gore locations. RPMs have been shown to reduce erratic maneuvers through painted gores at exits and bifurcations (Niessner, 1984). In another RPM study, Zwahlen (1987) evaluated various RPM spacings on freeway tangent sections and on ramps that were approximately 305 m (1,000 ft) long with a curvature of 24 degrees. The RPM spacings evaluated on the ramps were 3.8 m, 7.6 m, and 15.2 m (12.5 ft, 25 ft, and 50 ft) along the outer edgeline. These spacings were evaluated against a no-RPM condition. It was found that the addition of RPMs at any of the above spacings did not substantially improve driver performance. However, it must be recognized that the ramps on which the tests were conducted were of the cloverleaf type and, therefore, the exit speeds were most likely lower than can be expected on most two-lane rural roadways. Taylor and McGee (1973) also reported findings from past research on delineation of gore areas, demonstrating the effectiveness of RPMs.

The work by Taylor and McGee (1973) also included a comprehensive review of several case studies. As a result of their state-of-the-art summary, coupled with the results of their field observations in the study outlined above, a set of recommendations was developed for painted delineation, post delineators, and RPMs; these recommendations, which have since been widely implemented, are described below.

For painted delineation:

- 200- to 300-mm (8- to 12-in) wide white lines should be used to outline the exit gore, and where additional emphasis is necessary, diagonal or chevron markings are recommended.

- A 200-mm (8-in) wide line with a 1.5-m (5-ft) mark and 4.5-m (15-ft) gap should be used as an extension of the mainline right edgeline (or median edgeline for left exits) and should replace the lane line for at least 305 m (1,000 ft) upstream from the gore nose at an exit lane drop.

For post delineators:

- Post delineators should be placed in the gore area to enhance nighttime visibility. Crystal delineators are recommended for the through roadway side, and amber delineators should be used on the exit side. A spacing of 3–6 m (10–20 ft), depending on ramp divergence angle, is recommended.
- Amber delineators should be placed along the right edge of the deceleration lane at a spacing of 30.5 m (100 ft). Beyond the beginning of the gore, the spacing is dependent on the degree of curvature.
- Crystal delineators should be placed on the inside shoulder of the through roadway, at a spacing of 30.5 m (100 ft), to help strengthen the through-way delineation in the exit area.

For RPMs:

- Raised pavement markers are recommended as a supplement to standard gore paint markings and should be placed inside the "V" formed by the paint lines.
- Raised pavement markers should be supplemented with post delineators where the view of the roadway is limited, such as at vertical sections.

Hostetter, Crowley, Dauber, and Seguin (1989) conducted a controlled field study using 15 subjects ages 18–60 and older, to determine the effect of lighting, weather, and improved delineation on driver performance. Data were obtained on two exits in dry and wet weather under full lighting with baseline delineation (see diagram in Recommendation A(5)). The baseline system is similar to the delineation used at many of the partially lighted interchanges cataloged by the study authors during site selection, and in the opinion of an expert panel convened during the research, constituted a minimum system for partially lighted interchanges. Data were then obtained under partial lighting, with baseline and three improved delineation systems.

Upgrade 1 investigated by Hostetter et al. (1989) differed from the baseline in the use of RPMs along the left ramp stripe, and the substitution of fully retroreflective posts (117-cm [46-in] strip of 8-cm [3-in] wide sheeting) for partially reflective posts (46-cm [18-in] strip of 8-cm [3-in] wide sheeting) in the physical gore. Upgrade 2 differed from the baseline in the deployment of additional posts along the left ramp shoulder to create a spacing of 15 m (50 ft) rather than 30.5 m (100 ft) and in the installation of wide RPM's ("traffic diverters") on the gore strips to replace the 10-cm (4-in) RPM's placed adjacent to the gore stripes in the baseline system. Upgrade 3 replaced all baseline system partially retroreflective posts with fully retroreflective posts except in the gore, used RPM's along the left ramp stripe, and used beaded profiled tape containing a raised-diamond pattern for gore striping. The tape was used because it would project above a film of water during rain. The test sites were a half-diamond interchange and a full diamond which contained very little ramp curvature. The exit ramps were

4.3 m (14 ft) wide, with a single lane widening to two lanes near the intersection with the crossing roadways. Measures of effectiveness included ramp and spot/trap vehicle speeds, overall travel time, deceleration estimates, and lane placement, as well as selected types of erratic maneuvers and brake and high-beam headlight activations.

Analysis of delineation effects on ramp and spot speeds and on speed distributions showed few differences under dry conditions. Under rainy conditions, effects were stronger but were neither large enough nor consistent enough to recommend improved delineation over the baseline system. Although Upgrade 3 produced fewer edgeline encroachments under both dry and wet conditions, from the standpoint of operations, safety benefit, or cost-effectiveness, the upgrade did not demonstrate enough advantage to merit a recommendation for use on diamond interchanges with little ramp curvature.

B. Design Element: Acceleration/Deceleration Lane Design Features

Table 22. Cross-references of related entries for acceleration/deceleration lane design features.

Applications in Standard Reference Manuals
AASHTO Green Book (1994)
Pg. 573, Para. 4 Pgs. 941-942, Sect. on *Speed-Change Lanes* Pgs. 942-947, Sect. on *Single-Lane Free-Flow Terminals, Entrances*, Figures X-72 & X-73, and Tables X-4 & X-5 Pg. 952, Para(s). 2-7 Pg. 953, Figure X-75 Pg. 954, Para. 4 Pgs. 944 & 947—952, Sect. on *Single-Lane Free-Flow Terminals, Exits*, Figure X-74, and Tables X-5 & X-6 Pgs. 954-955, Para(s). 1-6 and Figure X-76 Pgs. 957-959, Sect. on *Two-Lane Entrances* and Figure X-80 Pgs. 958 and 960-961, Sect. on *Two-Lane Exits* and Figure X-81

Studies dating back to the 1960's have addressed the effects of ramp design on driving performance; however, Koepke (1993) reported that the basic design criteria, and therefore design standards, used by governmental agencies to design exit and entrance ramp terminals have not changed in more than 30 years. Recommendations for selected design features for interchange ramps may be justified by both the changing characteristics of the driving population and the operating characteristics of the highway system. Age-related functional decreases in visual acuity, motion judgment, and information-processing capabilities cause increased difficulty for older drivers entering and exiting highways. At the same time, traffic density has increased dramatically, resulting in more complex decisionmaking and divided attention requirements at these sites. In a survey of 664 drivers age 65 and older, one-half of those surveyed (49 percent) reported that the length of freeway entry lanes was a highway feature that was more important to them now compared with 10 years ago (Benekohal, Resende, Shim, Michaels, and Weeks, 1992).

The difficulties older drivers are likely to experience on freeway ramps, particularly acceleration lanes, are a function of changes in gap judgments resulting from a diminished capability to integrate speed and perceived distance information for moving targets; reduced neck/trunk flexibility; and age-related deficits in attention-sharing capabilities. First, the requirement to yield to approaching traffic on the mainline requires a merging driver to assess the adequacy of gaps in traffic by turning his/her head to look over the shoulder and/or by using the sideview mirrors. In a survey of 297 adults ranging in age from 22 to 92, which was conducted to gain a greater understanding of the visual difficulties they encounter while driving, the older participants reported greater difficulty judging both the speed of their vehicle and the speed of other vehicles, and expressed a concern over other vehicles "moving too quickly" (Kline, Kline, Fozard, Kosnik, Schieber, and Sekuler, 1992).

It has been shown that older persons require up to twice the rate of movement to perceive that an object is approaching, and require significantly longer to perceive that a vehicle is moving closer at a constant speed compared with younger individuals (Hills, 1975). Darzentas, McDowell, and Cooper (1980) used Hills' data in a simulation model to estimate conflict involvement for each class of subject as a function of main-road flow and speed. In the model, a conflict occurs when a poor gap acceptance decision is made by a driver, causing an oncoming vehicle to decelerate to avoid collision. Older drivers were involved in more conflicts than younger drivers of the same gender, and male drivers were involved in more conflicts than females in the same age class at all flows.

Other findings describing age differences in driver behavior on acceleration ramps are reported in a recent National Highway Traffic Safety Administration (NHTSA) study of driver age and mirror use. In this study, which measured the time required to make a "safe/unsafe" maneuver decision in a freeway lane-change situation, old-old drivers (age 75 and older) consistently required longer to make a lane-change decision than a group of drivers ages 65–74, who in turn demonstrated exaggerated response times compared with a younger control group (Staplin, Lococo, Sim, and Gish, 1996). This was a simulator study, using large screens showing dynamic videos of overtaking vehicles, in correct perspective, as the test stimuli; also, all drivers were forced to rely on their mirror information alone to make the maneuver decision in this research. The mean response time for a lane-change decision for the oldest (75 and older) driver group in this study, across a large number of trials in which the relative speed of the overtaking vehicle was varied between 16.1 and 40.25 km/h (10 and 25 mi/h) (i.e., faster than the subject's own vehicle was traveling when the video was shot), changed with changes in the target distance (separation of overtaking vehicle from driver). At close separation (30.5–61 m [100–200 ft]), where virtually all older drivers quickly decided that a lane-change maneuver was unsafe, decision latency averaged approximately 2.1 s. At a 61-m (200-ft) separation, some drivers were more willing to merge, and required longer to reach a maneuver decision, producing a mean latency of 2.5 s. At a 91.5-m (300-ft) separation and above (between the overtaking vehicle and the driver wishing to change lanes), maneuver decision latency reached an asymptote at 2.95 s, as increasing percentages of subjects accepted the available gap ahead of the overtaking vehicle.

Some relevant findings come from reviews of accident rates and ramp characteristics. Lundy (1967) found that off-ramp accident rates were consistently higher than on-ramp accident rates. However, Oppenlander and Dawson (1970) reported that at urban interchanges 68 percent of the interchange ramp accidents occurred at entrance ramps, while 32 percent occurred at exit ramps; for rural interchanges, these percentages were reversed. Similarly, Mullins and Keese (1961) reported that in urban areas, 82 percent of the interchange accidents occurred at on-ramps and 18 percent at exit ramps. Further, Lundy's (1967) study of 722 freeway ramps in California found that the accident rate was reduced for off-ramps when deceleration ramps were at least 274 m (900 ft) long (not including the length of the taper), for on-ramps when acceleration lanes were at least 244 m (800 ft) long, and for weaving sections that were at least 244 m (800 ft) long. Oppenlander and Dawson (1970) also concluded that safety was improved for on-ramps, off-ramps, and weaving areas 244 m (800 ft) in length or greater. Cirillo (1970) found that increasing the length of weaving areas reduced accident rates, and increasing the length of acceleration lanes reduced accident rates *if* merging vehicles constituted more than 6 percent of the mainline volume. Reduced accident rates from lengthening of *deceleration* lanes also appears to be related to the percentage of diverging traffic, with significant safety benefits beginning when 6 percent of the mainline traffic diverges (Cirillo, 1970).

The most comprehensive work to develop guidelines for freeway speed-change lanes (SCLs) was conducted in NCHRP project 3-35 by Reilly, Pfefer, Michaels, Polus, and Schoen (1989), who collected data on the entry and exit processes by videotaping 35 sites in three States. An entrance model was developed, based on gap acceptance and acceleration characteristics of drivers as determined by the controlling geometry. An exit model was developed, based on the driver's behavioral response to design geometrics. The purpose of the research was to develop new criteria which would offer greater flexibility than the (then) current AASHTO (1984) guidelines, which "do not provide the designer with the ability to reflect important geometric and traffic conditions" (Reilly et al., 1989). In this research, it was reported that the AASHTO (1984) SCL design criteria were based on the acceleration and deceleration characteristics of early-model vehicles, with little regard to traffic flow characteristics or driver behavior. The design values produced by the NCHRP project entry model for SCL length were slightly lower at low freeway speeds and significantly higher at moderate to high freeway speeds when compared with the 1984 AASHTO values. The exit model values for length were significantly higher than 1984 AASHTO values for all freeway and ramp speeds. The findings of the study suggest that for certain traffic conditions, the current SCL design criteria do not provide sufficient length for proper execution of the merge or diverge process. This is of particular importance with regard to the age-related diminished capabilities documented above.

In the consideration of *acceleration lanes and entrance ramps*, Michaels and Fazio (1989) reported on the model of freeway merging developed during the conduct of NCHRP project 3-35, to define SCL length. In this model, the merge process is composed of four sequential decision components, to which a fifth component is added: (1) a steering control zone (SC), which involves the steering and positioning of the vehicle along a path by steering from the controlling ramp curvature onto the SCL; (2) an initial acceleration zone (IA), in which the driver accelerates to reduce the speed differential between the ramp vehicle and the freeway vehicles to an acceptable level for completing the merge process; (3) a gap search and acceptance zone (GSA), during which the driver searches, evaluates, and accepts or rejects the available lags or gaps in the traffic stream; (4) a merge steering control zone (MSC), during which the driver enters the freeway and positions the vehicle in the nearest mainstream lane (Lane 1); and (5) a visual clear zone (VC), which provides a buffer between the driver and the end of the acceleration lane, where the driver can either merge onto the freeway in a forced maneuver or abort the merge and begin to decelerate at a reasonable rate. Associated with each of these components is a length; the total SCL length is the sum of the SC, IA, GSA, and VC components. The entry process is diagrammed in figure 12.

Design values for entrance ramp acceleration lane lengths were developed as a part of NCHRP 3-35 based on driver behavior and traffic flow characteristics obtained from field studies and known human factors. The model assumes that a driver will adopt a significant non-zero speed differential at the beginning of the GSA so as to facilitate entry into the traffic stream. In this model, it is recommended that a value of 16.1 km/h (10 mi/h) be used for that speed differential. In this research, it was found that it is not only the speed differential between the

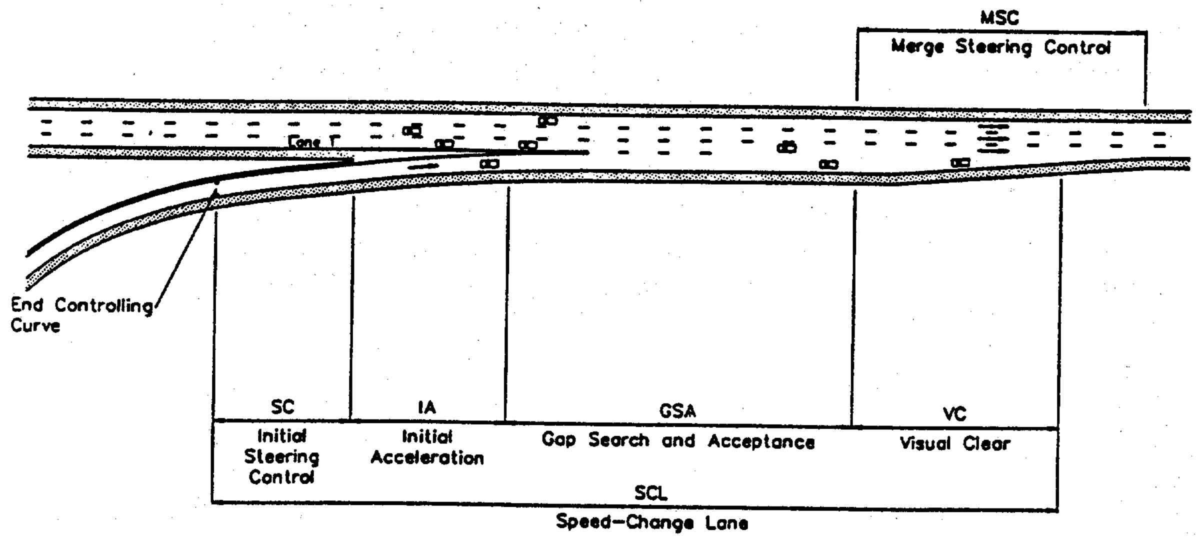


Figure 12. The entry process and components of the entry model developed in NCHRP 3-35.

ramp and freeway vehicles, but also the position of the vehicles relative to each other and the availability of a suitable gap in the freeway traffic, that determine when the merge will occur. The time for the SC is considered to be a constant, which is approximately 1 to 1.5 times the entry velocity, as it was estimated that a 1-s steering transition from ramp to acceleration lane would be sufficient. Therefore, at an entry speed of 15 m/s (50 ft/s), a maximum of 23 m (75 ft) should provide for the entry steering maneuver. The length of the acceleration segment (IA) depends on the magnitude of acceleration that is acceptable to the driver. If the driver accelerates at 1.5 m/s (4.8 ft/s) for only 2 s, he or she will have traveled 33.5 m (110 ft), which, when added to the steering control distance, means that the driver will have a clear view of oncoming traffic for a minimum of 49–56 m (160–185 ft). The appropriateness of these model assumptions for older drivers was not addressed in the NCHRP project, however.

As emphasized in NCHRP 3-35, the GSA is a key component of the entry model; this is especially true for older drivers. This length includes the distance required to search for and accept a headway, and is determined by the distribution of headways in Lane 1 of the freeway, the gap acceptance characteristics of the driver of the ramp vehicle, the design vehicle (car or truck), and the volume on the ramp. The angular velocity threshold—a critical variable because of its impact on GSA length and overall acceleration lane length—is set at 0.002 rad/s in the entry model. This value is based on field measurements and ensures that 85 percent of observed drivers in model validation studies (age not reported) will accept a gap producing an angular velocity of equal or greater value. The GSA length requires the use of 16 equations, which are documented in the NCHRP 3-35 report. There are a number of problems in applying these formulations using an older design driver, however. While it has been reported that drivers accept shorter gaps in freeway traffic than assumed by the model (Koepke, 1993), critical gap size for this as for other maneuvers increases significantly with increasing driver age. In addition, whereas Michaels and Fazio (1989) cited observed behavior whereby drivers judge gaps in sequence, increasing the probability of finding one acceptable by accelerating between successive searches, there is ample anecdotal evidence of older drivers slowing and often stopping in acceleration lanes when their initial search does not reveal an acceptable gap to merge with traffic on the mainline (Transportation Research Board, 1988). Finally, noting the increased reliance on mirror information for gap judgments in this situation by (older) persons with reduced neck/torso mobility, the exaggerated maneuver decision latencies in the Staplin et al. (1996) research on mirror-based lane change judgments reported earlier bear on GSA (and therefore, acceleration lane) length requirements.

The VC length is determined by the angular velocity of the target pavement area at the end of the ramp taper. It must provide the driver with sufficient distance to implement a forced merge or decelerate to a stop, to avoid running off the acceleration lane if he/she has not found an acceptable gap. In the model, if a driver on the acceleration lane is traveling at a speed of 21–24 m/s (70–80 ft/s), then as he/she approaches to within 61–76 m (200–250 ft) of the end of the lane or when the taper produces a lane width of less than 3 m (10 ft), the driver will begin to decelerate. Clearly, the delineation of the pavement width transition at the ramp terminus must be highly conspicuous, to accommodate older driver diminished visual capabilities.

Another issue addressed by NCHRP 3-35 was acceleration lane geometry. Koepke (1993) reported that 34 of the 45 States responding to a survey conducted as a part of NCHRP 3-35 on SCL's use a parallel design for entrance ramps. Thirty of the agencies interviewed use a taper design for exit ramps and a parallel design for entrance ramps. The parallel design requires a reverse-curve maneuver when merging or diverging, but provides the driver with the ability to obtain a full view of following traffic using the side and rearview mirrors (Koepke, 1993). Although the taper design reduces the amount of driver steering control and fits the direct path preferred by most drivers on *exit* ramps, the taper design used on entrance ramps requires multitask performance, as the driver shifts between accelerating, searching for an acceptable gap, and steering along the lane. Reilly et al. (1989) pointed out that the taper design for entrance lanes poses an inherent difficulty for the driver and is associated with more frequent forced merges than the parallel design. Forced merges were defined as any merge that resulted in the braking of lagging vehicles in Lane 1, or relatively quick lane changes by lagging vehicles from Lane 1 to a lane to the left. The parallel design would thus appear to offer strong advantages in the accommodation of older driver diminished capabilities.

In the consideration of *deceleration lanes and exit ramps*, Livneh, Polus, and Factor (1988) reported that studies analyzing traffic behavior on deceleration lanes have been few in number. They summarized Fukutome and Moskowitz's (1963) efforts to determine whether the length of the ramp tangent approaching the ramp curve had any effect on ramp speed. Fukutome and Moskowitz (1963) found that the length of the deceleration lane from the end of the taper should be at least 137 m (450 ft) when the ramp curve has a radius of 122 m (400 ft), and noted that shorter distances resulted in significantly lower speeds at the nose, which were reflected backward, causing interference to through traffic on the freeway. The results suggested that the shorter distances resulted in unnaturally high rates of deceleration, primarily affecting unfamiliar drivers who are more likely to have adjustment problems when unusual deceleration rates are applied. Fukutome and Moskowitz (1963) found that drivers prefer some moderate deceleration rate as opposed to an extremely low one afforded by a lengthy distance in which to accomplish the speed change. The design should allow the vehicle to enter the deceleration lane at a speed comparable to the through flow speed and decelerate in the deceleration area to the velocity required to negotiate the exit ramp properly.

As in the case of acceleration lanes, the speed-change maneuver on deceleration lanes was segmented into components in NCHRP 3-35 (Reilly et al., 1989). These components include: (1) the *diverge steering zone*, L_{DS}, which is the distance upstream from the exit gore at which a driver begins to diverge from the freeway; (2) the *steering control zone*, L_{SC}, in which the driver steers and positions a vehicle from the freeway lane onto the deceleration lane; (3) the *deceleration in-gear zone*, L_{DG}, in which the vehicle decelerates prior to braking; and (4) the *deceleration while braking zone*, L_{DB}, in which braking occurs in order to reach a reduced speed dictated by the geometrics, terminus, or traffic conditions on the off-ramp. The total deceleration lane length, L_{SCL}, is equal to $L_{SC} + L_{DG} + L_{DB}$. Figure 13 diagrams the exit process defined in the NCHRP research.

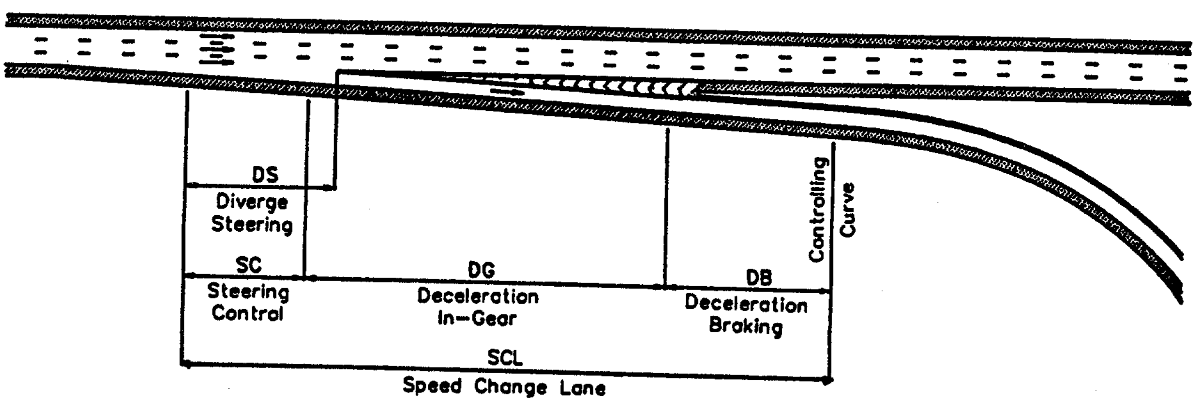


Figure 13. The exit process and components of the exit model developed in NCHRP 3-35.

The lengths of the four zones in the exit process were combined into two design elements: the L_{SCL}, which is the total length required to complete the exit process, and the L_{DS} , which defines the distance upstream from the nose of the exit wedge at which the beginning of the deceleration lane must be placed. Depending on the location of the speed-controlling point on the ramp, the driver will decelerate in-gear until the driver's angular velocity threshold has been reached and braking must occur. Therefore the total deceleration of the vehicle is a combined process between in-gear and braking. The length of the L_{DG} zone is the most sensitive to variations in diverge speeds; the L_{SC} and L_{DB} zones vary little with diverge speed. The design criteria for deceleration lanes are presented in *NCHRP 3-35 Speed-Change Lanes User Design Guidelines*; these criteria can be used to determine the required lengths for a new design, to test the appropriateness of an existing design, or to retrofit older designs not used by designers today.

A comparison of the values generated by the NCHRP exit model and current AASHTO values was presented by Reilly et al. (1989). For most freeway and ramp speeds, the model deceleration lane lengths are longer than the AASHTO values. The difference between the exit model and AASHTO values increases with increasing ramp speed.

The NCHRP model was validated using data observed at 12 sites. An assumption in the development of the exit model was that the speed of an exiting vehicle during the diverge steering maneuver is constant, and therefore the speed of the vehicle during the diverge equals the freeway speed. Data collected at 12 exiting sites during this study confirmed that the reduction in speed was normally less than 3.2 km/h (2 mi/h), regardless of the initial speed. However, it was found that a significant percentage of drivers reduce their speed while still on the freeway, prior to the diverge maneuver, with the average speed of 83.7 km/h (52 mi/h) across all sites prior to the diverge maneuver. Next, a critical element in the exit model is the angular velocity threshold, which determines L_{DS} and L_{DB}. As a driver approaches an exit, he/she first recognizes the taper diverging from the freeway lane, which is essentially a widening of the overall roadway. This recognition is determined mainly by the change in the driver's visual angle subtended by the roadway; however, other elements such as edge markings and signing will generate a component of angular velocity. In addition, the angular velocity will reach threshold at greater distances for a curved ramp than for a simple diverging ramp, resulting in the use of more deceleration lane length in cloverleaf interchanges than in diamond interchanges.

Complementing the findings in NCHRP 3-35, Livneh et al. (1988) observed traffic using freeway deceleration lanes at two freeway sites to record actual behavior and compare it to current design practice. They concluded that a considerable difference exists between the AASHTO assumptions and actual driver behavior along deceleration lanes. The principal discrepancies were in average speeds and in rate and duration of deceleration in-gear and while braking. The speed of both cars and heavy vehicles at the beginning of the deceleration lane was always lower than the average speed of through vehicles. The deceleration values obtained were lower than the values recommended by AASHTO. On properly designed long lanes, the duration and length of deceleration in-gear were longer than 3 s, as assumed by AASHTO, and deceleration-in gear took place for an average of 10 s until the speeds of vehicles slowed from about 85 percent of their average running speed on the through lane—the initial speed at the beginning of the taper—to an average of 67 percent. From this point, which was 200 m (650 ft) from the beginning of the deceleration lane, braking started and continued until speeds were further reduced to meet the average running speed required to negotiate safely the ramp curve that followed.

To meet the needs of older drivers, the point of controlling curvature on an exit ramp, as well as the curve speed advisory, must be highly conspicuous to create an appropriate expectancy of the required vehicle control actions. With this expectancy, older drivers should be able to negotiate deceleration lane geometries meeting AASHTO or NCHRP guidelines competently (also assuming effective gore delineation as discussed in *Handbook* section II-A). Raised curve delineation treatments may be recommended in this regard; post-mounted delineators or chevrons could be particularly effective. In addition, Holzmann and Marek (1993) noted that ramp operations may be improved by moving the relatively sharp ramp curvature away from the ramp terminal.

Finally, a recent review of interchange design issues, necessitated by changes in road user characteristics and current research, approached ramp geometry as a three-dimensional system (Keller, 1993). According to this review, the factors that influence ramp alignment and superelevation design include design consistency and simplicity, the roadway user, design speed, and (stopping and decision) sight distance. Because driver reaction time is slowed when elements of ramp geometry are different than expected, design should provide for long sight distances, careful coordination between horizontal and vertical alignment, generous curve radii, and smooth coordinated transitions, particularly when complex interchange designs are unavoidable. Increasing the sight distance and simplifying interchange layout can reduce some of the effects of decreasing visual acuity, short-term memory decline, reduced decisionmaking ability, reduced ability to judge vehicle speed, decreased muscle flexibility and pain associated with arthritis, and early fatigue and slower reaction times associated with increasing driver age. With regard to design speed, Keller (1993) stated that the ramp proper should be viewed as a transition area with a design speed equal to the speed of the higher speed terminal wherever feasible, and that few diagonal or loop ramps are long enough to accommodate more than two design speeds. Thus, the terminals and the ramp proper should be evaluated to determine the appropriate speed for design.

In terms of stopping sight distance (SSD) requirements, Keller (1993) noted that designers can reduce drivers' stress at interchanges by providing sight distances greater than the minimum SSDs. Although a brake reaction time of 2.5 s is representative of 90 percent of the drivers used in a 1971 study by Johansson and Rumar, and is used in the AASHTO SSD formula, it has been suggested that a 3.5-s perception and braking time should be used to accommodate the elderly with diminished visual, cognitive, and psychomotor capabilities (Gordon, McGee, and Hooper, 1984). Another assumption in the AASHTO calculations for SSD is a driver eye height of 1.06 m (3.5 ft); the eye height of older drivers is often less. Finally, alignment affects braking distance, such that curves impose greater demands on tire friction than tangents, resulting in increased braking distance when the friction requirements of curves and braking are combined (Glennon, Neuman, and Leisch, 1985).

Keller (1993) noted that locations where SSD values do not provide the time necessary to process information and react properly highlight the importance of the use of decision sight distance (DSD). Examples of locations at interchange ramps where DSD is desirable include ramp terminals at the main road, especially at an exit terminal beyond the grade separation and at left exits; ramp terminals at the cross road; lane drops; and abrupt or unusual alignment changes. AASHTO guidelines (1994) note that sight distance along a ramp should be at least as great as the safe stopping distance. The sight distance on a freeway preceding the approach nose of an exit ramp should exceed the minimum stopping distance for the through traffic speed, desirably by 25 percent or more, although the desirable goal remains DSD.

DSD values—which include detection, recognition, decision, and response initiation and maneuver times—are provided in AASHTO (1994) Table III-3 by design speed and type of avoidance maneuver required. Lerner, Huey, McGee, and Sullivan (1995) measured DSD for three driver age groups (ages 20–40, ages 65–69, and age 70 and older) at six freeway lane drop locations. While perception-reaction time values measured by Lerner et al. (1995) were actually somewhat lower than the values assumed by AASHTO, they nevertheless found that the 85th percentile *total* time required by each age group for detection, decision and maneuvering exceeded the recommended AASHTO value of 14.5 s. The freeway total times averaged 16.5 s, 17.6 s, and 18.8 s, for the three groups (from youngest to oldest), respectively. The researchers explained that the original AASHTO work assumed free-flow traffic conditions, in which drivers were not required to wait for a gap in traffic to change lanes. The Lerner et al. (1995) study, by comparison, was conducted on heavily traveled urban freeways, and subjects often had to wait for gaps in traffic before maneuvering. This led to significantly higher maneuver times than were assumed by AASHTO. No modifications to the existing DSD standards were deemed necessary. Keller (1993), reporting on the results of a 1991 survey about distances used when locating ramp exits beyond a crest vertical curve, indicated that 15 (38 percent) of State design agencies use the safe SSD, 9 (23 percent) use the safe SSD plus 25 percent, and 12 (31 percent) use DSD.

C. Design Element: Fixed Lighting Installations

Table 23. Cross-references of related entries for fixed lighting installations.

Applications in Standard Reference Manuals		
MUTCD (1988)	**AASHTO Green Book (1994)**	**Roadway Lighting Handbook Chapter 6 (1983)**
Pg. 2E-2, Para. 4 Pgs. 2F-7–2F-8, Sect. 2F-13	Pg. 310, Para. 2	Pgs. 16 - 23, Sect. on *Analytical Approach to Illumination Warrants* Pg. 20, Form 4 Pgs. 42-45, Sect. on *Summary of Light Sources* Pgs. 53-56, Sect. on *Classification of Luminaire Light Distributions* Pgs. 84-89, Sect. on *Interchange Lighting* Pgs. 120-139, Sect. on *Illumination Design Procedure*

Research has documented that: (1) freeway interchanges experience a higher accident rate than the mainline (Cirillo, 1968) and (2) urban freeway lighting has beneficial safety effects (Box, 1972). Cirillo (1968) also found a reduction in the number of interchange accidents as lighting intensity increased. Gramza, Hall, and Sampson (1980) evaluated the interchanges in the Interstate Accident Research (ISAR-2) database at which lighting had been introduced during the 10-year study period. During the daytime, there were 83 accidents before lighting and 80 accidents after lighting. At nighttime, by comparison, there were 76 accidents before lighting and 43 accidents after lighting. Taylor and McGee (1973) found a reduction in erratic maneuvers at exit lane drop sites in a before after study, when the exit area was illuminated during the "after" period of data collection.

Although nighttime driving is associated with a higher accident risk for drivers of all ages, the effects of aging on the visual system are further compounded by the effects of darkness. The aging process causes gradual declines in a variety of visual functions, including acuity, contrast sensitivity, glare recovery, and peripheral vision, making night driving especially difficult for older drivers. Of particular difficulty is the ability to notice and recognize objects at night and in low-light conditions such as dawn and dusk, rain, fog, haze, and snow. Between age 20 and age 70, aging directly reduces contrast sensitivity by a factor of about 3.0 (Blackwell and Blackwell, 1971); older drivers are thus at a greater relative disadvantage at lower luminance levels than younger drivers.

The impact for the older driver of lost sensitivity under nighttime conditions should be assessed against the nature of the night driving task. Even at night, most visual information is processed by the cone or daylight system in the foveal region of the retina where fine detail is resolved. Artificial lighting raises the illumination level of the roadway environment to the photopic range so that reading and tracking functions can occur. The peripheral rod system participates primarily by alerting the driver to a weaker signal away from the foveal line of sight, which may then be oriented to by the driver with a foveal fixation. The implication of a loss in rod sensitivity is that a much brighter peripheral signal will be needed to elicit proper visual attention from the driver, and that signals now falling below threshold will be ignored.

In fact, the signal may need to be 10 to as much as 100 times brighter, depending on age and object color (Staplin, Lococo, and Sim, 1990). Since both rod and cone thresholds increase with age, it is also true that more light will be needed to bring important tasks such as reading and tracking (path maintenance) above the cone limit.

There are a number of other aspects of vision and visual attention that relate to driving. In particular, saccadic fixation, useful field of view, detection of motion in depth, and detection of angular movement have been shown to be correlated with driving performance (see Bailey and Sheedy, 1988, for a review). As a group, however, these visual functions do not appear to have strong implications for highway lighting practice, with the possible exception of the useful field of view. It could be argued that it would be advantageous to provide wider angle lighting coverage to increase the total field of view of older drivers. High-mast lighting systems can increase the field of view from 30 degrees to about 105 degrees (Hans, 1993). Such wide angles of coverage might have advantages for older drivers in terms of peripheral object detection. However, while effective high-mast systems have been demonstrated (Ketvirtis and Moonah, 1995), such installations also tend to sacrifice target contrast for the increased field of view they provide.

The following paragraphs summarize studies that: (1) evaluated the effects of lighting on accident experience at interchanges and (2) evaluated specific aspects of driver performance as a function of number and type of luminaires at an interchange.

Gramza et al. (1980) conducted an accident analysis of 400 nighttime accidents that occurred at 116 interchanges during the period of 1971–1976, in 5 States (Maine, Maryland, Minnesota, Texas, and Utah). In an analysis of the presence of high-mast lighting at interchanges, versus no lighting or other kinds of interchange lighting, the presence of high-mast lighting was found to significantly reduce total accident rates, total accidents involving fatalities and injuries, and accidents involving fatalities and injuries other than the vehicle-to-vehicle and vehicle-to-fixed-object categories (e.g., accidents caused by striking pedestrians). Table 24, taken from Gramza et al. (1980), shows the predicted effect of high-mast lighting on annual number of accidents.

Table 24. Relative annual effect of lighting type on total nighttime accidents (n=400) at urban and nonurban interchanges. Source: Gramza, Hall, and Sampson, 1980.

Night Traffic Volume	Urban			Nonurban		
	Non-High-Mast	High-Mast	% Acc. Decrease	Non-High-Mast	High-Mast	% Acc. Decrease
5,000	2.0	0.0	100	3.6	0.4	89
7,500	3.8	0.6	84	5.4	2.2	59
10,000	5.7	2.5	56	7.3	4.1	44

Gramza et al. (1980) also found that although the number of lights at an interchange and the level of illumination had no significant effect on the *total* number of nighttime accidents, significant decreases in a variety of distinct accident types were found with increases in illumination. Increases in the illumination level—measured in lux or horizontal footcandles (hfc)—at interchanges were associated with significant reductions in two types of accidents: vehicle-to-fixed-object accidents involving property damage and vehicle-to-vehicle accidents involving fatalities and injuries. In addition, increases in the number of lights active at an interchange were found to significantly influence (reduce) the following two accident types: vehicle-to-fixed-object accidents involving fatalities and other injuries, and other property damage accidents. The number of lights at an interchange ranged from 0 to 114, with an average of 16 active lights and a median of 10. Thirty-two percent of the interchanges were unlit. As lighting levels increased, accident rates decreased. Illumination ranged from 0.0 lux to 10.76 lux (0.0 hfc to 1.0 hfc), with an average of 5.49 lux (0.51 hfc) for the lighted sections. These four accident types accounted for 61 percent of the accidents observed in the sample.

Since there were relatively few accidents per interchange per year, Gramza et al. (1980) employed a model to predict the number of each accident type per year, assuming 3 levels of traffic volume (average nighttime traffic of 5,000, 7,500, and 10,000 vehicles) at partial cloverleaf and other types of interchanges, and allowing varying levels of illumination or varying numbers of lights. The predicted relationships between traffic volume, lighting, and accident frequency showed that reductions in number of lights and in level of illumination (hfc) resulted in higher frequencies of vehicle-to-fixed-object and other property damage accidents, for all traffic volumes. Vehicle-to-vehicle accidents were also shown to increase in frequency as illumination was reduced, for all interchange types.

In addition, the findings at the level of one interchange were translated to estimate, as an overall annual impact for the five-State sample, the relative influence of the lighting variables on numbers of accidents at interchanges through three levels of night traffic volume. A level of 7.53 lux (0.7 hfc) was used to represent the allowable base of average maintained illumination. Overall, the model predicted that reductions in levels of illumination appear to cause greater increases in the numbers of accident types than do reductions in numbers of lights (Gramza et al., 1980).

Although the work of Gramza et al. (1980) is noteworthy in its attempt to quantify the complex relationships between interchange lighting and safety, it is critical to remember that their model was applied to data derived to fit 1975 conditions—including, by implication, both the then-current number of older drivers *and* their exposure to this highway feature during nighttime operations. By contrast, present and anticipated future driving patterns of older drivers—whose actual numbers as well as their percentage of all drivers will increase dramatically—show much higher use rates for freeways (Lerner and Ratté, 1991). This trend should sharply accentuate the safety impacts cited by Gramza et al.

Janoff, Freedman, and Decina (1982) conducted a study to determine the effectiveness of partial lighting of interchanges, where partial interchange lighting (PIL) was defined as lighting that consists of a few luminaires located in the general areas where entrance and exit ramps connect with the through traffic lanes of the freeway (between the gore and the end of the acceleration ramp/beginning of the deceleration ramp). A complete interchange lighting (CIL) system includes lighting on both the acceleration and deceleration areas plus the ramps through the terminus. In their survey of approximately 50 agencies which supplied information on over 14,000 interchanges and over 7,500 interchange lighting systems, it was found that 37 percent of the interchange lighting was CIL and 63 percent was PIL. An observational field study was conducted to determine the effects of lighting level (various levels of PIL, CIL, no lighting, and daylight), geometry of the interchange (straight versus curved ramps), and presence of weaving area versus no weaving area on driver behavior and traffic operations. PIL was stratified by the number of lights at each ramp, and included three levels: PIL 1 (one light), PIL 2 (two lights), and PIL 4 (four lights). CIL test sites included a full cloverleaf in suburban Baltimore, Maryland, and a three-leg interchange in suburban Philadelphia, Pennsylvania, with luminaire mounting heights of 12.2 and 9.5 m (40 and 31 ft), respectively. The dependent measures included speed and acceleration of individual vehicles traversing the interchanges; merge and diverge points of individual vehicles entering the main road or leaving it; and erratic maneuvers such as brake activations, use of high beams, and gore or shoulder encroachments.

Both field studies indicated that CIL provided a better traffic operating environment than did PIL and that any interchange lighting performed better than no lighting (although the differences were not always as great as between CIL and PIL). In particular, to the extent that traffic flow and safety are important issues, the Janoff et al. study concluded that existing CIL systems should not be reduced to PIL systems. When installing new lighting and economics are not an overriding issue, a CIL system is preferred over a PIL system. However, a PIL system with one or two luminaires per ramp will normally perform better than no lighting at far lower cost than a CIL system. PIL systems with fewer luminaires (one or two) frequently performed better than PIL systems with greater numbers of luminaires (four). This was explained by the fact that drivers may experience transitional visibility problems under the PIL conditions when they are forced to drive from dark to light to dark areas and at the same time perform complex maneuvers such as diverging, merging, and tracking a 90-degree curve.

Hostetter, Crowley, Dauber, and Seguin (1989) noted that when luminaires are not placed downstream of the physical gore of a partially lighted exit ramp, a driver proceeds from a lighted area to a nonlighted area. Citing evidence from various researchers (Boynton and Miller, 1963; Boynton, 1967; Boynton, Rinalducci, and Sternheim, 1969; Boynton, Corwin, and Sternheim, 1970; Rinalducci and Beare, 1974; and Fredericksen and Rotne, 1978), they reported that the effect of going from higher to lower levels of luminance results in a reduction in visual sensitivity, which would explain the findings of Janoff et al. (1982) that performance under partial lighting was better with fewer luminaires.

D. Design Element: Traffic Control Devices for Prohibited Movements on Freeway Ramps

Table 25. Cross-references of related entries for traffic control devices for prohibited movements on freeway ramps.

Applications in Standard Reference Manuals	
MUTCD (1988)	AASHTO Green Book (1994)
Pgs. 2E-22 to 2E-23 (Section 2E-40 "Wrong-Way Traffic Control") Pgs. 2E-25 to 2E-27 (figs. 2-22a, 2-22b, 2-22c)	Pgs. 914-915, entire section on "Wrong-Way Entrances"

It has been reported that out of 100 wrong-way accidents, 62.7 result in an injury or fatality, versus 44.2 out of 100 for all freeway or expressway accidents (Tamburri and Theobald, 1965). These data highlight the fact that wrong-way accidents are more severe than most other types. The most frequent origin of wrong-way incidents, as reported by these authors, was entering the freeway via an off-ramp.

Results of more recent investigations of the wrong-way problem in California indicate that fatal wrong-way accidents as a percentage of all fatal accidents on freeways have decreased substantially in the last 20 years (Copelan, 1989). The actual number of wrong-way fatal accidents was the same in 1987 as it was in 1963 (about 35 per year), despite the fact that freeway travel has increased fivefold; the reduction appears to be related to the countermeasures employed by California Department of Transportation over the intervening years, including the implementation of guide and wrong-way signs and pavement markings providing better visual cues. Copelan (1989), while noting that half of the wrong-way driving on freeways was from deliberate, illegal U-turns, reported that additional improvements could still significantly reduce wrong-way accidents. In their study of highway information systems, Woods, Rowan, and Johnson (1970) found that motorists frequently experience difficulty in locating entrance ramps to freeways, and drivers were often confused when there were several side roadways intersecting in close proximity to the interchange area. These researchers suggested that more efficient use could be made of "positive" signing techniques in guiding motorists to the freeway entrance ramps and discouraging drivers from possible wrong-way maneuvers.

Early studies found that the rate of wrong-way driving based on vehicle-miles of travel increased with driver age (Tamburri and Theobald, 1965). In their analysis of 1,214 wrong-way driving incidents which occurred over 2 9-month periods on California highways, they found a moderate increase in incidents for drivers ages 30–39 and those ages 40–49. Over age 60, the incidents rose rapidly; and over age 70, incidents occurred at rates approximately 10 times higher than for drivers ages 16–29. Lew (1971) reported on an analysis of 168 wrong-way accidents by civilians on California freeways in which the age of the wrong-way driver was recorded. While certain age groups (i.e., 30–39, 50–59, and 60–69) were represented to an extent corresponding closely to their proportion of the driving population, other groups such as those ages 16–19, 40–49, and 70–79 deviated markedly from expectation. Drivers ages 16–19 experienced approximately one-half of the wrong-way accidents expected for their age group;

drivers ages 40–49 experienced three-quarters of the rate expected; and drivers ages 70–79 experienced *over twice* the number of freeway wrong-way accidents than would be expected.

Age-related diminished capabilities contributing to wrong-way movements include the cognitive capabilities of selective and divided attention, and the sensory/perceptual capabilities of visual acuity and contrast sensitivity. Selective attention refers to the ability to identify and allocate attention to the most relevant targets in the driving scenario on an instant-to-instant basis, while divided attention refers to the ability to perform multiple tasks simultaneously. Individuals less capable of switching attention, or who switch too slowly, may increase their chances of choosing the wrong response or choosing the correct response too slowly. Treat, Tumbas, McDonald, Shinar, Hume, Mayer, Stansifer, and Castellan (1977) reported that 41 percent of accidents in which older adults were involved were caused by a failure to recognize hazards and problems, and that 18 to 23 percent of their accidents were due to problems with visual search. The selective attention literature generally suggests that for adults of all ages, but perhaps particularly for the elderly, the most relevant information must be signaled in a dramatic manner to ensure that it receives a high priority for processing in situations where there is a great deal of complexity at the level of information to be processed.

Older drivers' use of signs designed to control wrong-way movements is affected by their visual performance capabilities. Letter acuity declines during adulthood (Pitts, 1982) and older adults' loss in acuity is accentuated under conditions of low contrast, low luminance, and high visual complexity. A field investigation (Sivak, Olson, and Pastalan, 1981) of the effect of driver's age on nighttime legibility of highway signs indicated that older subjects perform substantially worse than younger subjects on a nighttime legibility task using a wide range of currently available sign materials.

Aside from difficulties in the use of signs, problems for older drivers at interchanges most likely to result from (age-related) deficits in spatial vision relate to the timely detection and recognition of pavement markings and delineation. Data from a study by Blackwell and Blackwell (1971) show that between age 20 and age 70, aging directly reduces contrast sensitivity by a factor of about 3.0. Mace (1988) stated that age differences in glare sensitivity and restricted peripheral vision coupled with the process of selective attention may cause higher conspicuity thresholds for older drivers. Overall, these deficits point to the need for more effective and more conspicuous signing and delineation.

Violations of driver expectancy, use of alcohol, and reductions in the ability to integrate information from multiple sources to make navigation decisions while concurrently controlling the vehicle may all result in driver confusion at critical decision points, resulting in wrong-way maneuvers. Tamburri and Theobald (1965) found that many older drivers and drinking drivers did not know where their wrong-way movement began (i.e., they could identify neither where the decision point was nor the location of the wrong-way maneuver).

Vaswani (1974) identified specific sources of wrong-way movements where alcohol was believed *not* to be a factor. In this study, exit ramps on partial interchanges generated wrong-way maneuvers because, unlike the ramps on full interchanges that converge with right-hand traffic, the ramps meet the crossroad at about 90 degrees to accommodate both left and right turns. Therefore the wrong-way entries consist of left turns off of the exit ramp into wrong-way traffic on a two-way divided highway, right turns from the divided highway into traffic exiting the ramp, and left turns from the crossroad into the exit ramp. At intersections with four-lane divided highways (divided arterial and primary highways), 45 percent of the wrong-way entries were at their intersections with exit ramps or secondary roads. The wrong-way entries were due to left-turning vehicles making an early left turn rather than turning around the nose of the median. Almost all these accidents involved sober drivers.

Some ramp designs are more problematic than others. In Tamburri and Theobald's 1965 analysis of 400 wrong-way incidents where entry was made to the freeway via an off-ramp, the trumpet interchange category had the highest wrong-way entry rate, with 14.19 incidents per 100 ramp-years, and the full cloverleaf interchanges had the lowest wrong-way entry rate, with 2.00 incidents per 100 ramp-years. Parsonson and Marks (1979) also determined that several ramp types were particularly susceptible to wrong-way movements, as follows: half-diamond (3.9 per month), partial cloverleaf ("parclo") loop ramp (11.0 per month) and parclo AB loop ramp (6.7 per month). The parclo loop ramp and the parclo AB loop ramp share the same problem, which is an entrance and exit ramp in close proximity. The half-diamond is susceptible because it is an incomplete interchange, and drivers may make intentional wrong-way entries. A "problem" ramp has been defined as one that experiences more than five wrong-way movements per month; a corrected ramp has less than two per month (Rinde, 1978).

Preventative measures for reducing the frequency and severity of wrong-way maneuvers include modifications in ramp and roadway geometry, and signing and pavement markings, and the use of warning and detection devices and vehicle arresting systems. Selected countermeasures are discussed below.

Vaswani (1974) found that on almost all the interchanges on which wrong-way entries had been made into the exit ramp or from the exit ramp onto the crossroad, the corner of the exit ramp flared into the right pavement edge of the crossroad. He suggested that such a flare provides for a very easy but incorrect right-hand turn, and may help to induce a driver to make a wrong-way entry from the crossroad into the exit lane. A countermeasure consisting of a sharp right-hand junction would require a driver to reduce speed and almost come to a stop before maneuvering into the left lane, and would also reduce the chances that a driver exiting the ramp would turn left into wrong-way traffic on the crossroad. Site inspections showed that where the flare was not provided and the left lane of the exit ramp and the passage through the median were channelized, no wrong-way entry to or egress from the exit ramps was reported. Additionally, Vaswani (1974) reported that generous widths of an exit ramp with its junction with the crossroad make wrong-way entry or egress from the exit ramp easy. Narrow pavement widths will discourage such entries. A serious impediment to turning maneuvers by heavy vehicles could also result from this strategy, however.

Vaswani (1974) also indicated that too large a set-back of the median noses from the exit ramp increases the width of the crossover and makes the intersection harder to "read." Vaswani suggests that if the width cannot be reduced, then pavement nose markings in the form of a striped median should be applied, for improved visibility of this design element.

Campbell and Middlebrooks (1988), following the recommendation of Parsonson and Marks (1979) to widely separate the on- and off-ramps at partial cloverleaf interchanges, experimented with a design in which close exit and entrance ramps would be combined into one paved surface separated only by a double yellow line. Ten ramps in the Atlanta, Georgia, area were redesigned and evaluated using actual counts of wrong-way movements. Two of the ramps were monitored before and after they were converted to combined ramps. At the first location, the wrong-way rate per month before construction was 86.7; after combining the ramps, the rate fell to 0.3 per month. At the second location, the wrong-way rate was 88.6 per month. After the installation of four countermeasures (trailblazers, lowered DO NOT ENTER and WRONG WAY signs, 450-mm [18-in] stop bar, and 200-mm [8-in] yellow ceramic buttons in the centerline of the crossroad), the rate dropped to 2.0 per month. Once the ramps were combined at this second location, the wrong-way rate jumped to 30.0 per month, even when ceramic buttons, permanent signing, and pavement markings and a dotted channelizing line (i.e., paint stripes that lead turning vehicles onto the ramp) were employed.

The mixed results of the Campbell and Middlebrooks study (1988) led to the evaluation of 15 additional combined ramps in the same research project, 12 of which were partial cloverleaf, with the balance consisting of median entrance/exit ramps (designed for future access by high-occupancy vehicles to the median lanes, but during the study period were open to all traffic). The study periods ranged from 30 to 102 days. The results clearly indicated that the concept of combined exit and entrance ramps can work when signing and markings conform to MUTCD specifications. It was recommended that 200-mm (8-in) yellow ceramic buttons be installed along the cross street centerline if all other countermeasures do not work.

With regard to signing, Woods et al. (1970) indicated that positive signing which indicates the correct path or turning maneuver to the motorist rather than a restriction may help most to minimize driver confusion at freeway interchanges. Examples include route markers, trailblazers, and a FREEWAY ENTRANCE sign that positively designates an entrance to the freeway. Friebele, Messer, and Dudek (1971) noted that the use of oversized signs and reflectorization may be needed in locations where motorists are apt to disregard wrong-way warnings, and Copelan (1989) suggested that the larger, highly reflective signs may be helpful for confused or elderly drivers.

Parsonson and Marks (1979) found that lowering the DO NOT ENTER and WRONG WAY signs to 450 mm (18 in) above the pavement to place them in the path of the headlight beams at night and placing trailblazer signs on the on-ramp were effective, inexpensive countermeasures. Individually, these two countermeasures reduced the wrong-way incidence to about one-third to one-half of its original rate. This is consistent with California's Standard Sign Package, which specifies that the DO NOT ENTER and FREEWAY ENTRANCE packages be mounted with the bottom of the lower sign 600 mm (24 in) above the edge of the pavement. It also specifies that ONE WAY arrows be mounted 450 mm (18 in) above the pavement. The Virginia Department of Highways and Transportation (1981b) noted concern regarding the 450 mm (18 in) mounting height of the ONE WAY signs, however, stating that the signs may become obscured by vegetation and by guardrails (when the sign is mounted behind a guardrail). Thus, mounting height was revised for this State to 900 mm (36 in), to alleviate these concerns.

California uses the DO NOT ENTER and WRONG WAY signs together on a single signpost, with the WRONG WAY sign mounted directly beneath the DO NOT ENTER sign (the Do Not Enter Package). This sign package is placed on *both* sides of the ramp. The California Standard specifies that large FREEWAY ENTRANCE signs (1,200 mm x 750 mm [48 in x 30 in]) be placed on on-ramps, but the location of the sign package (FREEWAY ENTRANCE sign, plus route shield, cardinal direction sign, and down diagonal arrows) should not be controlled by the use of the larger signs; smaller signs (900 mm x 525 mm [36 in x 21 in]) may be used for proper placement, if necessary. For off-ramp signing, the Standard specifies the use of at least one Do Not Enter package (DO NOT ENTER and WRONG WAY signs), to be placed to fall within the area covered by the car's headlights and visible to the driver from the decision point on each likely approach; three or four packages may be required if the off-ramp is split by a traffic island. In addition, ONE WAY arrows should be placed as close to the crossing street as possible. The MUTCD standard sizes for the DO NOT ENTER and WRONG WAY signs are 750 mm x 750 mm (30 in x 30 in) and 900 mm x 600 mm (36 in x 24 in), respectively. California uses sizes of 900 mm x 900 mm; 1,200 mm x 1,200 mm; and 1,800 mm x 1,800 mm (36 in x 36 in; 48 in x 48 in; and 72 in x 72 in) for the DO NOT ENTER sign and 900 mm x 525 mm and 1,800 mm x 525 mm (36 in x 21 in and 72 in x 21 in) for the WRONG WAY sign. As they are retrofitted and newly installed, the Do Not Enter sign packages in California have high-intensity sheeting (Copelan, 1989).

Turning to a consideration of pavement markings, Tamburri (1969) found that a white pavement arrow placed at all off-ramps pointing in the direction of the right-way movement can be effective in reducing the number of wrong-way maneuvers. However, Parsonson and Marks (1979) found that at a parclo AB loop off-ramp that has its crossroad terminal adjacent to the on-ramp, standard pavement arrows, lowered DO NOT ENTER and WRONG WAY signs, trailblazer signs, and a 600-mm- (24-in)-wide painted stop bar were not sufficient, as the ramp still showed 22.3 wrong-way movements per month. Large pavement arrows (7.3 m [24 ft long]) and yellow ceramic buttons (200 mm [8 inches] in diameter] to form a median divider on the crossroad were required, in addition. It was specified that the ceramic buttons should touch each other to form a continuous, unbroken barrier, and should extend far enough toward the interchange structure (the freeway) to prevent a wrong-way driver from avoiding the buttons by turning early. The length required is typically 30.5 m (100 ft). The addition of the ceramic buttons reduced wrong-way maneuvers from a rate of 88.6 per month to a rate of 2.0 per month. Campbell and Middlebrooks (1988) also found that installing yellow ceramic buttons to the extension of the centerline of the crossroad to aid in channelizing left-turning traffic entering the freeway, in combination with countermeasures employed by the Georgia Department of Transportation as standard practice—trailblazer sign, 450-mm- (18-in)-wide stop line at the end of the off-ramp, 5.5-m- (18-ft)-long painted pavement arrow, and lowered WRONG WAY and DO NOT ENTER signs—reduced wrong-way maneuvers. It was also recommended in the Parsonson and Marks (1979) study that the two-piece, 7.3-m- (24-ft)-long painted arrow pavement marking (part of the California standard, described by Gabriel, 1974, and depicted in Parsonson and Marks) be adopted.

With regard to other pavement marking countermeasures, Copelan (1989) reported that red, airport-type pavement lights installed across an off-ramp, which became "activated" by the headlights of a wrong-way vehicle, were effective in reducing wrong-way freeway entries. In this observational study of seven off-ramps in San Diego, California, that were determined to have operational problems (e.g., a history of wrong-way entrances and/or misleading layout of ramps), approximately one-half of the (potential) wrong-way drivers applied their brakes before reaching the WRONG WAY signs, and one-half of the drivers continued past the signs, but applied their brakes before reaching the pavement lights.

III. ROADWAY CURVATURE AND PASSING ZONES

The following discussion presents the rationale and supporting evidence for *Handbook* recommendations pertaining to these four design elements (A–D):

A. Pavement Markings and Delineation on Horizontal Curves
B. Pavement Width on Horizontal Curves
C. Crest Vertical Curve Length and Advance Signing for Sight Restricted Locations
D. Passing Zone Length, Passing Sight Distance, and Passing/Overtaking Lanes on Two-Lane Highways

A. Design Element: Pavement Markings and Delineation on Horizontal Curves

Table 26. Cross-references of related entries for pavement markings and delineation on horizontal curves.

Applications in Standard Reference Manuals	
MUTCD (1988) Rev Part VI (1993)	AASHTO Green Book (1994)
Pg. 3A-2, Last para. Pg. 3B-21, Para(s). 2 & 3 (Item 1) Pg. 3B-22, Para(s). 1, 2, & 5 Pg. 3D-1, Para.1 Pg. 3D-2, Para. 8 Pg. 3D-3, Para. 4 & Table III-1 Pg. 6D-3, Item 5 Pg. 71, Para(s). 3 (Item 6) and 4 Pg. 73, Para. 2	Pg. 43, Para. 6 Pg. 44, Para(s). 1 & 6 Pg. 45, Para. 2 Pg. 46, Para. 3 Pg. 52, Para. 2 Pg 314, Para(s). 6 & 7 Pg. 315, Para. 3

Note: Page letter references (e.g., 3A-2) refer to the MUTCD (1988), while those with only numbers (e.g., Pg. 71) refer to Rev Part VI of the MUTCD (1993).

Pavement markings and delineation devices serve important path guidance functions on horizontal curves, particularly under adverse visibility conditions, at twilight, and at nighttime. They provide a preview of roadway features ahead and give the driver information about the vehicle's lateral position on the roadway. Delineation must provide information that results in recognition of the boundaries of the traveled way both at "long" preview distances (5 to 8 s of travel time) and at more immediate proximities (within 1 s of travel time) where attention is directed toward instant-to-instant vehicle control responses.

Surface pavement markings in current practice may vary along four dimensions: brightness, width, thickness, and the addition of structure to "thick" applications. Stripes of increased thickness have an advantage in wet weather because the material is more likely to protrude above the level of surface water and to provide a degree of retroreflectivity greater than that provided by thinner applications of paint. Also, the commercially available structured stripes (tapes) are brighter than other marking treatments, even under dry conditions. This is due to the ability of the raised element of the structure to reflect more light back to the driver than a horizontal surface. Even greater benefits are provided by reflectorized treatments, including raised pavement markers (RPM's), post-mounted delineators (PMD's), and chevron signs, which may be used to improve the nighttime visibility of delineation and to indicate roadway alignment.

A number of driver visual functions that have an impact on the use of pavement markings and delineation show significant age-related decrements: dynamic acuity, contrast sensitivity, dark adaptation, and glare recovery. Dynamic visual acuity (DVA) includes the ability to resolve the details of a high-contrast target that is moving relative to an observer. Activities that rely on dynamic acuity include making lateral lane changes and locating road boundaries when negotiating a turn. In these situations, greater speeds are associated with poorer DVA. Contrast sensitivity influences the response to both sharply defined, bright versus dark visual targets, and those with grayer, less distinct edges. In general, older adults tend to have decreased contrast sensitivity (Owsley, Sekuler, and Siemsen, 1983). This loss is more pronounced at lower light levels (Sloane, Owsley, and Alvarez, 1988; Sloane, Owsley, and Jackson, 1988) and is associated with a heightened sensitivity to glare (Wolf, 1960; Fisher and Christie, 1965; Pulling, Wolf, Sturgis, Vaillancourt, and Dolliver, 1980). The findings of Blackwell and Blackwell (1971) indicate that a 60-year-old observer needs approximately 2.5 times the contrast as a 23-year-old observer for the same level of visibility.

Highway research studies that have varied one or more of the four dimensions of pavement markings are discussed below, along with studies on the effectiveness of RPM's, PMD's, chevron signs, and combinations of delineation treatments. Age differences are reported wherever data are available.

An early study of surface pavement markings employing an interactive driving simulator, plus field evaluations, concluded that driver performance—measured by the probability of exceeding lane limits—was optimized when the perceived brightness contrast between pavement markings and the roadway was 2.0 (Blackwell and Taylor, 1969). A study by Allen, O'Hanlon, and McRuer (1977) also concluded that delineation contrast should be maintained above a value of 2.0 for adequate steering performance under clear night driving conditions. In other words, these studies have asserted that markings must appear to be at least three times as bright as the road surface, because contrast is defined as the difference between target and background luminance, divided by the background luminance alone. A difficulty with these studies, however, is that their data were not derived from—and thus are not representative of—normatively aged older drivers. The ideal viewing conditions assumed by Allen et al. (1977) also disregard the effects of glare as well as adverse visibility, and both factors have a disproportionate impact on the performance of older drivers. In Blackwell and Taylor's work, a minimum preview time of 3 to 4 s was recommended for accurate maneuvering under adverse conditions. However, more conservative estimates of preview time to accommodate older drivers (e.g., 5 s) have frequently appeared in the literature.

Freedman, Staplin, Gilfillan, and Byrnes (1988) showed significant performance decrements for 65-year-old drivers, as compared with 35-year-old drivers, in the visibility distance of 100-mm (4-in) pavement stripes on a simulated wet roadway. Staplin, Lococo, and Sim (1990) confirmed the need for higher levels of line brightness for older drivers in a simulator study, where the contrast for a 100-mm (4-in) white edgeline was continuously varied within a 40-step range in a method of limits. Under simulated opposing headlamp glare conditions, subjects ages 65–80 required an increase in contrast of 20 to 30 percent over a younger sample to correctly discern downstream curve direction at criterion viewing distances. To accommodate less capable older drivers, this study's results indicated that an increase in stripe brightness that is tenfold greater (300 percent) for older versus younger drivers may be warranted.

To describe the magnitude of the effects of age and visual ability on delineation detection/recognition distance and retroreflective requirements for threshold detection of pavement markings, a series of analyses using the Ford Motor Company PC DETECT computer model (cf. Matle and Bhise, 1984) yielded the stripe contrast requirements shown earlier in this *Handbook* in table 9 for Design Element F (Edge Treatments/Delineation of Curbs, Medians, and Obstacles) in the Rationale and Supporting Evidence section for Intersections (At-Grade). PC DETECT is a headlamp seeing-distance model which uses the Blackwell and Blackwell (1971, 1980) human contrast sensitivity formulations to calculate the distance at which various types of targets illuminated by headlamps first become visible to approaching drivers, with and without glare from opposing headlights. The top 5 percent (most capable) of 25-year-olds and bottom 5 percent (least capable) of 75-year-olds were compared in this analysis.

The more realistic operating conditions modeled as described above, together with the widely cited multiplier for older observers advocated in the seminal work by Blackwell and Blackwell (1971), support the recommendation that an in-service pavement edge striping contrast value on horizontal curves maintained at or above 5.0 is appropriate to accommodate the needs of the large majority of older drivers on highways and arterials without median separation between opposing directions of traffic. Where a median barrier (e.g., Jersey barrier) high enough to shield drivers from direct view of oncoming headlights is present, or where median width exceeds 15 m (49 ft), a horizontal curve edgeline contrast value of 3.75 or higher is recommended. It is important to note that these recommendations assume the standard stripe width of 100 mm (4 in). Where wider pavement markings are implemented, either as general or spot treatments, the same contrast values apply. It may be inferred from various studies of stripe width (e.g., Good and Baxter, 1986; Deacon, 1988) that treatments that are maintained at or above the recommended contrast levels *and* are wider than 100 mm (4 in) will provide the greatest benefit to older drivers. Contrast remains the preeminent factor in stripe visibility, however, and increased width alone does not substitute for lower-than-recommended contrast levels.

Raised pavement markers have received widespread use because they provide better long-range delineation than conventional painted lines, particularly under wet conditions. When used on a road edge, they also provide brighter peripheral cues, which could be advantageous to the older driver for path guidance. Over time, however, RPM's also are subject to loss of their initial retroreflectivity; in colder climates, RPM's may be damaged by plowing operations.

Deacon (1988), in his review of research on delineation and marking treatments that he believed would be of particular benefit to the older driver, found that highways with RPM centerlines had lower crash rates than those with painted centerlines. The average reduction in crash rates was approximately 0.5 crashes per million vehicle-miles. Zador, Stein, Wright, and Hall (1986) observed that after-modification vehicle paths were shifted away from the centerline on right and left curves with RPM's mounted on both sides of the double yellow centerlines, and that placement changes were largest with RPM's compared with PMD's and chevrons. It has also been observed that RPM's placed in the centerlines and edgelines at pavement width reductions at narrow bridges produce significant reductions in 85th percentile speeds and centerline encroachments (Niessner, 1984). On two-lane rural curves, RPM's in conjunction with the double yellow centerline have been recommended.

An RPM spacing study was conducted by Blaauw (1985), who tested several RPM patterns on 200-m (656-ft) radii and 1,000-m (3,281-ft) radii horizontal curves using a visual occlusion technique. White RPM's were used for the tests, at spacing distances of approximately 12.2 m, 24.4 m, and 36.6 m (40 ft, 80 ft, and 120 ft). On 200-m (656-ft) radius curves, the 24.4-m and 36.6-m (80-ft and 120-ft) spacings led to speed reductions and lane errors. Based on these results, it was recommended that on curves of this severity, the spacing of RPM's be restricted to 12.2-m (40-ft) spacings. In general, no differences between treatments were observed for the more gentle, 1,000-m (3,281-ft) radius curves.

Accordingly, this *Handbook* includes a recommendation for RPM installation, at standard (12.2-m [40-ft]) spacings, on all horizontal curves with radii below 1,000 m (3,281 ft).

Roadside delineators and treatment combinations are also important to this discussion. Because of its increasing use throughout the United States, and because it accommodates different types of sheeting in varying amounts and different designs, the primary roadside delineation device of current interest is the flat, flexible post. The general accident data have shown that the installation of PMD's is associated with lower crash rates for highway sections with or without edgelines (Bali, Potts, Fee, Taylor, and Glennon, 1978; Schwab and Capelle, 1979). Deacon (1988) confirmed that installation of PMD's lowered crash rates, for sections with or without edgelines. The reduction in crash rates resulting from the installation of these delineators averaged 1.0 crashes per million vehicle-miles. Thus, especially for lower functional classification roadways where the use of enhanced (e.g., wider) edgelines may be limited (due to pavement width restrictions), existing data suggest that PMD's can be an effective countermeasure.

In a driver performance study evaluating the effects of chevron signs, PMD's, and RPM's, both Johnson (1984) and Jennings (1984) found that driver performance on sharp curves was the most favorable when chevrons were used. With chevrons, drivers followed a better path around the curve (defined in terms of the ratio of the vehicle's instantaneous radius to the actual curve radius). These studies also revealed that drivers use a corner-cutting strategy, and that chevron signs and PMD's facilitated this strategy. On right curves with chevrons, drivers had an average midcurve placement closest to the centerline. On left curves with chevrons, vehicle placement was not significantly different. In the Good and Baxter (1986) study, chevron signs had a detrimental effect on control behavior, but were rated favorably by drivers in reducing task difficulty. Zador et al. (1986) found that chevrons (as well as RPM's) tend to shift vehicles away from the centerline on right and left curves, while PMD's shift vehicles away from the centerline on right curves. A particular advantage for chevrons with high-intensity reflective sheeting was demonstrated for drivers age 65 and older in a study by Pietrucha, Hostetter, Staplin, and Obermeyer (1994), when used in combination with other treatments.

The Pietrucha et al. (1994) study was specifically directed to the difficulties older drivers have with horizontal curve delineation elements, and the possible benefits of brighter materials, larger target sizes, redundant and/or multidimensional cues using combinations of elements, and novel designs or configurations of elements. Twenty-five distinct delineation/pavement marking treatments (a baseline treatment and 24 enhancements) were initially presented to subjects in 3 driver age groups (18–45, 65–74, and 75 and older). The baseline treatment was a 100-mm (4-in) yellow centerline at in-service brightness level (ISBL). The 24 treatments varied according to the presence/absence of edgeline, edgeline width, whether the edgeline was enhanced with RPM's, whether the centerline was enhanced with RPM's, and the presence/absence of off-road elements and their characteristics (material, color, brightness, and/or spacing). Measures of effectiveness were downstream roadway feature recognition (subjects were required to report the direction in which the roadway curved) and recognition distance in a 35-mm simulation of nighttime driving. Treatments that included the addition of RPM's to *both* the centerline and edgeline, and all treatments that included delineating the roadway edge with high-intensity chevrons or high-intensity PMD's, resulted in significantly higher mean recognition distances when compared with the baseline treatment, across all age groups. For the subjects age 65 and older, only a subset of the treatments with delineated roadway edges resulted in significantly higher mean recognition distances, due to the increased variance among older subjects' data. Next, field evaluations were conducted with a subset of the most promising treatments. The treatment with the highest recognition distance for both age groups consisted of a 100-mm (4-in) yellow centerline at ISBL with yellow RPM's at ISBL and standard spacing, a 100-mm- (4-in)- wide white edgeline, and fully reflectorized T-post delineators with standard spacing. For the 152.4-m (500-ft) radius of curvature used in this study, spacing for the PMD's was 19.8 m (65 ft). This treatment included PMD's that were fully reflectorized, i.e., retroreflective material extended from the top of the post to the ground and provided more reflective area than the standard posts most frequently used.

Blaauw (1985) tested combinations of PMD's and RPM's, resulting in the following recommendations: (1) RPM's exclusively at the center are favorable for lateral vehicle control inside the lane (short-range delineation) but are less adequate for preview information on the lane to be followed (long-range delineation); therefore, it is necessary to delineate both lane boundaries; (2) effective centerline delineation can be realized with RPM's; (3) delineation at the outside of the traffic lane can be realized with RPM's at the location of the lane boundary or with PMD's spaced laterally at 1.5 m (5 ft)—both configurations are equally efficient, but PMD's at an approximate 3.7-m (12-ft) spacing are less efficient; and (4) RPM's at the location of the center and/or lane boundaries must be applied with a maximum spacing distance of 12 m (40 ft) on a curve with 200-m (656-ft) radius or less.

In a laboratory study of drivers' responses to videotapes of four rural horizontal curves, six levels of delineation plus two levels of curvature were studied by Rockwell and Smith (1985). The levels included no delineation; centerline only; centerline plus edgeline; centerline plus edgeline plus PMD's; centerline plus edgeline plus RPM's; and centerline plus edgeline plus PMD's plus RPM's. Subjects were required to identify precisely the instant that they could detect the presence of a curve and then express their level of confidence with their response. The largest increase in detection distance was associated with the addition of RPM's and PMD's to the centerline and edgeline treatments, respectively.

While no *specific* roadside treatment on horizontal curves is advocated in this *Handbook*, a recommendation for roadside delineation devices at minimum spacings keyed to curve radius appears justified by the findings reported above. Using current practice as a guide, a spacing of 12 m (40 ft) represents an average value in table III-1 of the MUTCD, *Suggested Spacing for Highway Delineators on Horizontal Curves*, for curves with radii from 15 to 150 m (50 to 500 ft). This value is also consistent with the 12-m (40-ft) spacing requirement for RPM's on curves with radii ≤ 200 m (656 ft) noted above.

B. Design Element: Pavement Width on Horizontal Curves

Table 27. Cross-references of related entries for pavement width on horizontal curves.

Applications in Standard Reference Manuals
AASHTO Green Book (1994)
Pg. 43, Para. 6 Pg. 44, Para. 1 Pg. 83, Para. 3 Pg. 212, Para. 3 Pg. 213, Para. 2 Pgs. 214-219, Sect(s). on *Derivation of Design Values; Design Values; and Attainment of Widening on Curves.*

Roadway alignment is a key factor in unsafe vehicular operation: i.e., increasing degrees of curvature cause more accidents (Haywood, 1980). The widening of lanes through horizontal curves, minimizing the use of controlling or maximum curvature for a given design speed, and the use of special transition curves for higher speed and sharper curve designs have all been suggested as countermeasures. Whereas in the past lane widening has been advocated to accommodate the tracking of large trucks through curves, the present focus is on the accommodation of older drivers, whose diminished physical and perceptual abilities make curve negotiation more difficult. Lane widths on horizontal curves range from 2.7 m to 4 m (9 ft to 13 ft), but are usually 3.4 m or 3.7 m (11 ft or 12 ft) wide. Neuman (1992) recommended that when less than 3.7-m- (12-ft)-wide lanes are used, consideration should be given to widening the lane to this dimension through horizontal curves; and a further increase in width of 0.3–0.6 m (1–2 ft) may be advised to provide for an additional margin of safety through the curve for heavy vehicles. This margin of safety could also be justified in terms of its benefit to older drivers with diminished physical abilities.

Older drivers, as a result of age-related declines in motor ability, have been found to be deficient in coordinations involved in lanekeeping, maintaining speed, and handling curves (Brainin, Bloom, Breedlove, and Edwards, 1977). McKnight and Stewart (1990) also reported that older drivers have difficulty in lanekeeping, which results in frequently exceeding lane boundaries, particularly on curves. Drivers who lack the required strength, including older drivers and physically limited drivers, often swing too wide in order to lengthen the turning radius and minimize rotation of the steering wheel.

Joint flexibility is an essential component of driving skill. Osteoarthritis, the most common musculoskeletal disability among older individuals, affects more than 50 percent of the population age 65 and older (Roberts and Roberts, 1993). If upper extremity range of movement is impaired in the older driver, mobility and coordination are seriously weakened. Older drivers with some upper extremity dysfunction may not be able to steer effectively with both hands gripping the steering wheel rim. In a study of 83 people with arthritis, 7 percent used the right hand only to steer and 10 percent used only the left hand (Cornwell, 1987).

The general relationship between pavement width and safe driving operations has been well documented. Choueiri and Lamm (1987) reported the results of several early studies that found an association between decreasing accident frequency and increasing pavement widths. Krebs and Kloeckner (1977) reported that for every 1-m (3.3-ft) increase in pavement width, a decrease of 0.25 in the accident rate (per million vehicle-kilometers) could be expected. Hall, Burton, Coppage and Dickinson (1976) examined the nature of single-vehicle accidents involving fixed objects along the roadside of nonfreeway facilities. They found that the majority of these types of accidents were reported as nonintersection related, and occurred most frequently on weekends, at night, under adverse pavement and weather conditions, and on horizontal curves (especially outside of curve). These accident types have high injury severity to drivers and passengers. Wright and Robertson (1979) reported that 40 and 31 percent of all fatal crashes in Pennsylvania and Maryland, respectively, resulted in a vehicle hitting a fixed object such as a tree, utility pole, or bridge abutment. In a study focused on 600 accident sites (and 600 comparison sites) involving fixed objects, crash locations were best discriminated from comparison locations by a combination of curvature greater than 9 degrees and downhill gradient steeper than 3 percent; and, for the fatal fixed-object crash population, the crash locations were best discriminated from comparison locations by a combination of curvature greater than 6 degrees and downhill gradient steeper than 2 percent.

Glennon and Weaver (1971) evaluated the adequacy of geometric design standards for highway curves by filming vehicles entering unspiraled highway curves with curvature ranging from 2 to 7 degrees. While driver age was not analyzed, results of the study indicated that most vehicle paths, regardless of speed, exceed the degree of highway curve at some point on the curve. Glennon, Neuman, and Leisch (1985) measured vehicle speed and lateral placement on horizontal curves and found that drivers tend to overshoot the curve radius, producing minimum vehicle path radii sharper than the highway curve, and that the tendency to overshoot is independent of speed. They observed that the tangent alignment immediately in advance of the curve is the critical region of operations, because at about 61 m (200 ft) before the beginning points of the curve (or approximately 3 s driving time), drivers begin to adjust both their speed and path. Such adjustments are particularly large on sharper curves. Thus, the margin of safety in current AASHTO design policy is much lower than anticipated.

Zegeer, Stewart, Reinfurt, Council, Neuman, Hamilton, Miller, and Hunter (1990) conducted a study to determine the horizontal curve features that affect accident experience on two-lane rural roads and to evaluate geometric improvements for safety upgrading. An analysis of 104 fatal and 104 nonfatal accidents on rural curves in North Carolina showed that in more of the fatal accidents, the first maneuver was toward the outside of the curve (77 percent of the fatal accidents versus 64 percent of the nonfatal accidents). For approximately 28 percent of the fatal accidents (versus 8.8 percent of the nonfatal accidents), the vehicle ran off the road to the right and then returned to be involved in a crash. Further, an analysis on 10,900 horizontal curves in the State of Washington with corresponding accident, geometric, traffic, and roadway data variables showed that the percentages of severe nonfatal injuries and fatalities were greater on curves than on tangents with the same width, where total road width (lanes plus shoulders) was ≤ 9 m (30 ft).

Zegeer et al. (1990) concluded that widening lanes or shoulders on curves can reduce curve accidents by as much as 33 percent. Specifically, table 28 shows the predicted percent reduction in accidents that would be expected on horizontal curves by widening the lanes and by widening paved and unpaved shoulders (Zegeer et al., 1990).

Table 28. Percent reduction in accidents on horizontal curves with 2.4 m (8 ft) beginning lane width as a result of lane widening, paved shoulder widening, and unpaved shoulder widening. Source: Zegeer et al., 1990.

Total Amount of Lane or Shoulder Widening (ft)		Percent Accident Reduction		
Total	Per Side	Lane Widening*	Paved Shoulder Widening	Unpaved Shoulder Widening
2	1	5	4	3
4	2	12	8	7
6	3	17	12	10
8	4	21	15	13
10	5	*	19	16
12	6	*	21	18
14	7	*	25	21
16	8	*	28	24
18	9	*	31	26
20	10	*	33	29

1 ft = 0.305 m

* Values of lane widening correspond to a maximum widening of 8 ft (2.4 m) to 12 ft (3.7 m) for a total of 4 ft (1.2 m) per lane, or a total of 8 ft (2.4 m) of widening.

The evidence cited above from the engineering studies describing curve negotiation, pavement width, and accident reduction, together with the documented difficulties in lanekeeping and diminished motor abilities of older drivers, support the recommendation for a minimum pavement width (including shoulder) of 5.5 m (18 ft) on arterial horizontal curves over 3 degrees of curvature (cf. Cirillo and Council, 1986). It is understood that limited-access highways already exceed this recommended lane-plus-shoulder width. However, older drivers often report a preference to travel on two-lane arterials, and these facilities may be deficient in this regard, especially in rural settings.

C. Design Element: Crest Vertical Curve Length and Advance Signing for Sight-Restricted Locations

Table 29. Cross-references of related entries for crest vertical curve length and advance signing for sight-restricted locations.

Applications in Standard Reference Manuals	
MUTCD (1988) Rev Part VI (1993)	AASHTO Green Book (1994)
Pgs. 2-C & 2C-2a, Sect. on *Placement of Warning Signs* Pg. 2C-22, Para. 1 Pg. 40, Para(s). 5-7 Pg. 41, Entire Page	Pg. 46, Para. 1 Pgs. 283-286, Sect. on *Design Controls - Stopping Sight Distance* Pg. 314, Para. 2

Note: Page letter references (e.g., 2C-22) refer to the MUTCD (1988), while those with only numbers (e.g., Pg. 40) refer to Rev Part VI of the MUTCD (1993).

From a human factors perspective, the accommodation of older driver needs should be a high priority at sight-restricted locations because of the potential for violation of expectancy, even though the actual percentage of accidents occurring under conditions of limited (vertical) sight distance is quite small (Pline, 1996). Older drivers, as a result of their length of experience, develop strong expectations about where and when they will encounter roadway hazards and "high-demand" situations with increased potential for conflict. At the same time, older driver reaction time is slower in response to unexpected information, and older drivers are slower to override an initial incorrect response with the correct response. Further, aging is associated with physical changes that may interfere with rapid vehicle control when an emergency maneuver is required.

Of greatest importance during the approach to sight-restricted locations are the cognitive components of driving, most notably selective attention and response speed (complex reaction time). Selective attention refers to the ability to identify and allocate attention appropriately to the most relevant targets at any given time (Plude and Hoyer, 1985). One important finding in the selective attention literature, as noted above, is that older adults respond much more slowly to stimuli that are unexpected (Hoyer and Familant, 1987), suggesting that older adults might be particularly disadvantaged when an unexpected hazard appears in the road ahead. In fact, Stansifer and Castellan (1977) suggested that hazard recognition errors can be interpreted more as attention failures than as sensory deficiencies. The selective attention literature suggests that for adults of all ages, but perhaps particularly for the elderly, the most relevant information should be signaled in a dramatic manner to ensure that it receives a high priority for processing.

Next, appropriate vehicle control behaviors when unexpected hazards are encountered depend upon "speeded responding," or how quickly an individual is able to respond to a relevant target, once identified. A timely braking response when one recognizes that the car ahead is stopped or that a red signal or STOP sign is present can determine whether or not there is a crash. Thus, reaction time or the ability to respond quickly to a stimulus is a critical aspect of successful driving. Mihal and Barrett (1976) measured simple, choice, and complex reaction time and reported that simple and choice reaction time were not correlated with accidents, but complex reaction time was. Moreover, when only older adults were examined, the correlation with accident involvement increased from 0.27 for complex reaction for the total sample to 0.52, suggesting the relationship to be particularly marked for older adults. There is nearly uniform agreement among researchers that reaction time (speed) decreases with age. In particular, studies have demonstrated a significant and disproportionate slowing of response for older adults versus young and middle-aged adults as uncertainty level increased for response preparation tasks. Preparatory intervals and length of precue viewing times appear to be crucial determinants of age-related differences in movement preparation and planning (Eisdorfer, 1975; Stelmach, Goggin, and Garcia-Colera, 1987; Goggin, Stelmach, and Amrhein, 1989).

In summary, the age-related deficits in reaction time and various aspects of attention are not independent of one another, and more than one of these mechanisms is likely to reduce driving efficiency in the older adult. Because of these deficits, sight-restricted locations pose a particular risk to older drivers, presenting a need for recommendations addressing both geometry and signing that can be reconciled with available highway research findings in this area.

Unfortunately, there is a lack of conclusive data on this subject. Kostyniuk and Cleveland (1986) analyzed the accident histories of 10 matched pairs of sites on 2-lane rural roadways. The 10 limited sight distance (vertical curve) locations were defined as those below the minimum stopping sight distance (SSD) recommended by AASHTO in 1965, and ranged from 36 m to 94 m (118 ft to 308 ft). The control site locations were defined as those that more than met the standard (SSD greater than 213 m [700 ft]). Seven of the limited sight distance sites had more accidents than the matched control sites, two were approximately equal, and one had fewer accidents (Pline, 1996). Overall, the set of sites with less than minimum SSD had over 50 percent more accidents in the study period than the control sites.

Farber (1982) performed sensitivity analyses of the effects of change in eye height, object height, friction, and speed on SSD on crest vertical curves. He found that SSD was relatively insensitive to a reasonable range of changes in driver eye height, but was very sensitive to speed, friction, and reaction time. Thus, stopping distance on vertical curves that are of inadequate length or are substandard according to other design criteria, and where major redesign, repaving, or excavation is not feasible, could most efficiently be made safer by modifying a driver's approach speed and/or reaction time. For 88 km/h (55 mi/h) traffic, stopping distance increases 24.7 m (81 ft) for every 1-s increase in reaction time. Similarly, stopping distance decreases about 4.9 m for each 1-km/h reduction in speed (or 26 ft for each 1 mi/h). A need for more effective traffic control countermeasures is thus highlighted.

A reevaluation of crest vertical curve length requirements was performed by Khasnabis and Tad (1983). These researchers reviewed the historical changes in parameters that affect the computation of SSD and evaluated the effect of these changes on the length requirements of crest vertical curves. Principal conclusions were that further tests on reaction time are needed, since the current 2.5-s reaction time may not accurately reflect the changing age distribution and composition of the driving population. In addition, the validity of the assumption of a speed differential for wet pavement conditions between design speed and top driving speed is questionable, since there is very little evidence to substantiate the assumption that all motorists are likely to reduce their speed on wet pavements. Of particular interest, Khasnabis and Tad (1983) noted that the object height of 150 mm (6 in) appears to be somewhat arbitrary (i.e., the current AASHTO design criterion), and stated that reducing the object height to 75 mm (3 in) would improve the safety elements of crest curves.

In contrast, there are strong proponents of the position that the obstacle height criterion for design of vertical curves should be *raised* to 450 mm (18 in), or the approximate height of a passenger vehicle's rear taillights (see Neuman, 1989). While McGee (1995) has reported that available data are insufficient to definitively establish the relationship between (limitations in) vertical alignment and highway safety, and there is an indisputable logic in using a height criterion corresponding to the most commonly encountered obstacle on the road (i.e., another vehicle), this approach disproportionately penalizes older drivers in those rare circumstances when a hazard (of any type) appears unexpectedly due to sight-restricting geometry. Also, the simple argument that a conclusive relationship cannot be demonstrated as justification for changing current practice is somewhat disingenuous—a significant relationship between visual acuity and accident involvement has proven elusive, over decades of study, yet there is widespread acknowledgment that good vision is necessary for safe driving.

Returning to a consideration of potential countermeasures, as stopping distance is sensitive to decreases in speed and reaction time, any traffic control device that lowers either parameter is beneficial. In one study, a LIMITED SIGHT DISTANCE (W14-4) sign with a speed advisory was found to be understood by only 17 percent of the 631 respondents who passed through the study sight (Christian, Barnack, and Karoly, 1981). Part of the problem may be that unlike the hazards cited by other warning signs, the phrase "limited sight distance" has no tangible manifestation, and even when drivers have topped the crest of a vertical curve, they may not be aware of the extent to which their sight distance was reduced. Freedman, Staplin, Decina, and Farber (1984) developed and tested the effectiveness of both verbal and symbol alternative warning devices for use on crest vertical curves using drivers ages 16–75. The existing LIMITED SIGHT DISTANCE sign, with or without a supplementary speed advisory panel, did not produce desirable driver responses (braking or slowing) as frequently, nor was it recalled, comprehended, recognized, or preferred as often as the verbal alternative SLOW HILL BLOCKS VIEW sign, or an alternative symbol sign that depicted two vehicles approaching from opposite sides of a hill.

With a focus on the conspicuity and legibility of static warning signs (i.e., as may be placed in advance of sight-restricted locations), a survey by the American Automobile Association Foundation for Traffic Safety found that 25 percent of older drivers experienced problems reading traffic signs (Yee, 1985). Olson and Bernstein (1979) suggested that older drivers should not be expected to achieve an legibility index (LI) of 0.6 m/mm (50 ft/in) under most nighttime circumstances. The data provided by this report give some expectation that 0.48 m/mm (40 ft/in) is a reasonable goal under most conditions for an "average" driver, that is one whose performance is at the 50th percentile (median) level for his or her age. To accommodate less capable older drivers, an LI of less than 0.48 m/mm (40 ft/in) would be of clear benefit. Larger sign panel sizes may be required to accommodate the larger characters necessary to achieve this LI for some messages.

Several studies have shown that the use of active sign elements, such as flashing warning lights for SLOW WHEN FLASHING and MAX SPEED ___MPH messages supplementing various standard warning signs, increases the conspicuity of the signs and results in greater speed reductions (Zegeer, 1975; Hanscomb, 1976; Lanman, Lum, and Lyles, 1979; Lyles, 1981) as well as a 60–70 percent reduction of accidents at grade crossings compared with the static sign alone conditions (Hopkins and Holmstrom, 1976; Hopkins and White, 1977). According to Pline (1996), several agencies have experienced success with the use of flasher-augmented warning signs with the legend PREPARE TO STOP when there is limited sight distance to a signalized intersection, activated at the time of signal change (red phase).

Lyles (1980) compared the effects of warning signs at horizontal and crest vertical curves with limited sight distance (less than 152.4 m [500 ft]). Five warning devices were evaluated: (1) the standard intersection crossroad warning symbol sign; (2) a warning sign with the message VEHICLES ENTERING; (3) a sequence of two warning signs and a regulatory sign (REDUCED SPEED AHEAD, crossroad symbol, and 35 mph speed limit sign); (4) a VEHICLES ENTERING sign with constantly flashing warning lights; and (5) the same as (4) but with a WHEN FLASHING plate, with flashing warning lights activated only in the presence of crossroad traffic. Overall, the standard crossroads and VEHICLES ENTERING signs had less speed-reducing effect (0.8–3.2 km/h [0.5–2 mi/h]) than the warning-warning-regulatory sequence and the signs with warning lights (6.4–8 km/h [4–5 mi/h]). This trend was the same for both horizontal and vertical curves, and there was no significant difference between the warning-warning-regulatory sequence and the signs with warning lights. Motorists were twice as likely to recall the warning-warning-regulatory sequence and signs with warning lights than the standard signs, and a van positioned at the crossroad was also reported to have been seen more often with these sign configurations.

As reviewed above, studies have shown that, in general, approach speeds to crest vertical curves make safe response by older drivers to a revealed obstacle unlikely given current design criteria. There is ample evidence of significant age-related declines in response capability to unexpected hazards. Analyses of curve length requirements conclude that safety benefits will result from a lower object height, yet practical considerations have prompted a move toward a higher criterion. Retention of the 150-mm (6-in) criterion is the most prudent practice to preserve existing levels of safety, as a steadily increasing segment of the driving population experiences diminished capability in terms of a number of relevant aspects of response effectiveness. In addition, conspicuous and comprehensible warning devices should be especially beneficial to elderly drivers in sight-restricted situations. Accordingly, a preservation of highway design adequacy and an improvement in motorist information are the goals of the recommendations in this section.

D. Design Element: Passing Zone Length, Passing Sight Distance, and Passing/Overtaking Lanes on Two-Lane Highways

Table 30. Cross-references of related entries for passing zone length, passing sight distance, and passing/overtaking lanes on two-lane highways.

Applications in Standard Reference Manuals	
MUTCD (1988)	AASHTO Green Book (1994)
Pg. 2C-4, Para(s) 4 & 5 Pg. 2C-21, Para. 2	Pg. 44, para. 3 Pgs. 128-134, Sect(s). on *Passing Sight Distance for Two-Lane Highways*

The safety and effectiveness of passing zones depend upon the specific geometric characteristics of the highway section, as well as on how drivers receive and process information provided by signs and pavement markings, integrate speed and distance information for opposing vehicles, and control their vehicles (brake and accelerate) during passing maneuvers. As the number of older drivers in the population increases dramatically over the years 1995–2025, many situations are expected to arise where not only the slower-moving vehicle, but also the passing vehicle, is driven by an older person.

The capabilities and behavior of older drivers, in fact, vary with respect to younger drivers in several ways crucial to this discussion. Studies have shown that while driving speed decreases with driver age, the sizes of acceptable headways and gaps tend to increase with age. While motivational factors (e.g., sensation seeking, risk taking) have been shown to play a major role in influencing the higher speeds and shorter headways accepted by young drivers, they seem to play a less important role in older driver behavior. Instead, the relatively slower speeds and longer headways and gaps accepted by older drivers have been attributed to their compensating for decrements in cognitive and sensory abilities (Case, Hulbert, and Beers, 1970; Planek and Overend, 1973).

The ability to judge gaps when passing in an oncoming lane is especially important. For some older drivers, the ability to judge gaps in relation to vehicle speed and distance is diminished (McKnight and Stewart, 1990). Depth perception—i.e., the ability to judge the distance, and changes in distance, of an object—decreases with age (Bell, Wolfe, and Bernholtz, 1972; Henderson and Burg, 1973, 1974; Shinar and Eberhard, 1976). A recent study indicated that the angle of stereopsis (seconds of visual arc) required for a group of drivers age 75 and older to discriminate depth using a commercial vision tester was roughly twice as large as that needed for a group of drivers ages 18–55 to achieve the same level of performance (Staplin, Lococo, and Sim, 1993). McKnight and Stewart (1990) reported that the inability to judge gaps is not necessarily associated with a high accident rate, to the extent that drivers can compensate for their deficiencies by accepting only inordinately large gaps. This tactic has a negative impact on operations as traffic volumes increase, however, and may not always be a feasible approach.

Judging in-depth motion is made difficult by the fact that when no lateral displacement occurs, the primary depth cue is the expansion or contraction of the image size of the oncoming vehicles (Hills, 1980). Studies of crossing-path crashes, where gap judgments of oncoming vehicle speed and distance are critical as in passing situations, indicate an age-related difficulty in the ability to detect angular movement. In laboratory studies, older persons required significantly longer to perceive that a vehicle was moving closer (Hills, 1975). Staplin and Lyles (1991) reported research showing that, relative to younger drivers, older ones underestimate the speed of approaching vehicles. Similarly, Scialfa, Guzy, Leibowitz, Garvey, and Tyrrell (1991) showed that older adults tend to overestimate approaching vehicle velocities at lower speeds and underestimate at higher speeds, relative to younger adults. Older persons also apparently accept a gap to cross in front of an oncoming vehicle that is a more-or-less constant distance, regardless of the vehicle's speed. Analyses of judgments of the "last possible safe moment" to cross in front of an oncoming vehicle showed that older men accepted a gap to cross at an average constant distance, whereas younger men allowed a constant time gap and thus increased distance at higher speeds (Hills and Johnson, 1980). A controlled field study showed that older drivers waiting (stationary) to turn left at an intersection accepted the same size gap regardless of the speed of the oncoming vehicle (48 km/h and 96.5 km/h [30 mi/h and 60 mi/h]), while younger drivers accepted a gap that was 25 percent larger for a vehicle traveling at 96.5 km/h (60 mi/h) than their gap for a vehicle traveling at 48 km/h (30 mi/h) (Staplin et al., 1993).

Consistent with the AASHTO operational model (AASHTO, 1994), passing sight distance is provided only at places where combinations of alignment and profile do not require the use of crest vertical curves. For horizontal curves, the minimum passing sight distance for a two-lane road is about four times as great as the minimum stopping sight distance at the same speed (AASHTO, 1994). By comparison, the MUTCD defines passing sight distance for vertical curves as the distance at which an object 1,070 mm (3.5 ft) above the pavement surface can be seen from a point 1,070 mm (3.5 ft) above the pavement. For horizontal curves, passing sight distance is defined by the MUTCD as the distance measured along the centerline between two points 1,070 mm (3.5 ft) above the pavement on a line tangent to the embankment or other obstruction that cuts off the view of the inside curve (MUTCD, 1988). The length of passing zones or the minimum distance between successive no-passing zones is specified as 122 m (400 ft) in the MUTCD. As Hughes, Joshua, and McGee (1992) pointed out, the MUTCD sight distance requirements were based on a "compromise between a delayed and a flying passing maneuver," traceable back to the AASHTO 1940 policy that reflected a "compromise distance based on a passing maneuver such that the frequency of maneuvers requiring shorter distances was not great enough to seriously impair the usefulness of the highway."

The basis for the minimum length of a passing zone (122 m [400 ft]) is unknown, however, because research has indicated that for design speeds above 48 km/h (30 mi/h) the distance required for one vehicle to pass another is much longer than 122 m (400 ft) (Hughes et al., 1992). Weaver and Glennon (1972) reported that, in limited studies of short passing sections on main rural highways, most drivers do not complete a pass even within a 244-m (800-ft) section; and use of passing zones remains very low when their length is shorter than 274.3 m (900 ft). Not surprisingly, it has been mentioned in the literature (Hughes et al., 1992) that the current AASHTO and MUTCD passing sight distance values are probably too low. Several studies have indicated that both the MUTCD and AASHTO passing sight distances are too short to allow passenger cars to pass trucks and for trucks to pass trucks (Donaldson, 1986; Fancher, 1986; Khasnabis, 1986).

Several research studies have been performed that have established and evaluated passing sight distance values for *tangent* sections of highways. As early as 1934, the National Bureau of Standards measured the time required for passing on level highways during light traffic and found that the time to complete the maneuver always ranged between 5 and 7 s regardless of speed. Passing maneuvers were observed at speeds ranging from 16 to 80 km/h (10 to 50 mi/h). They concluded that 274.3 m (900 ft) of sight distance was required for passing at 64 km/h (40 mi/h). Harwood and Glennon (1976) reported that drivers are reluctant to use passing zones under 268 m (880 ft). They recommended that design and marking standards should be identical and include both minimum passing sight distances and minimum length of passing zones, with minimum passing sight distance values falling between the AASHTO and MUTCD values. Kaub (1990) presented a substantial amount of data on passing maneuvers on a recreational two-lane, two-way highway in northern Wisconsin. Under low and high traffic volumes, he found that 24–35 percent and 24–50 percent, respectively, of all passes were attempted in the presence of an opposing vehicle; the average time in the opposing lane (96 km/h [60 mi/h]) was 12.2 s under low-traffic conditions and 11.3 s with high-traffic volumes.

Passing lanes, also referred to as overtaking lanes, are auxiliary lanes provided on two-lane highways to enhance overtaking opportunities. Harwood, Hoban, and Warren (1988) reported that passing lanes provide an effective method for improving traffic operations problems resulting from the lack of passing opportunities due to limited sight distance and heavy oncoming traffic volumes. In addition, passing lanes can be provided at a lower cost than that required for constructing a four-lane highway. Based on Morrall and Hoban (1985), the design of overtaking lanes should include advance notification of the overtaking lane; a KEEP RIGHT UNLESS OVERTAKING sign at the diverge point; advance notification of the merge and signs at the merge; and some identification for traffic in the opposing lane that they are facing an overtaking lane. They reported that there is general agreement that providing short overtaking lanes at regular spacing is more cost-effective than providing a few long passing lanes. This feature becomes increasingly attractive as the diversity of driving styles and driver capability levels grows, with faster motorists taking unnecessary chances to overtake slower-moving vehicles.

Finally, although the minimum passing sight distances specified by AASHTO are more than double that specified by the MUTCD, and are based on observations of successful car-passing-car observations, Hughes et al. (1992) commented that the model does not take into account the abortive passing maneuver, nor does it consider the length of the impeding vehicle. Saito (1984) determined that the values specified by the MUTCD for minimum passing distance are inadequate for the abortive maneuver, while Ohene and Ardekani (1988) asserted that the MUTCD sight distance requirements are adequate for the driver to abort if the driver decelerates at a rate of 3.2 m/s^2 for a 64-km/h passing speed (10.5 ft/s^2 for a 40-mi/h passing speed) and at a rate of 3.9 m/s^2 for a passing speed of 80 km/h (12.8 ft/s^2 for a 50-mi/h passing speed). Worth noting is work by Lyles (1981) on passing zone traffic control devices showing that aborted passes could be reduced by more judicious use of passing zone signs. In any event, it cannot be assumed that drivers will always use the maximum acceleration and deceleration capabilities of their vehicles, particularly older drivers.

The age differences in driver capability and behavior noted earlier—i.e., age-related difficulties in judging gaps and in increased perception-reaction time, coupled with slower driving speeds—support a recommendation for passing zone length that is consistent with the upper range of the time and distance values for passing maneuvers reported in this discussion. A recommendation for minimum passing sight distance (MUTCD definition), by comparison, may be keyed to the time required to perceive the need and execute appropriate vehicle control movements to abort a passing maneuver and return to one's own lane. This distance may therefore be smaller than the minimum passing zone length, but should allow an exaggerated perception-reaction time (5 s) to accommodate age-related declines in depth perception and a sufficient interval (3 s) for a smooth lane-change maneuver at passing speeds up to 96 km/h (60 mi/h). In addition, it appears reasonable to recommend a treatment to improve drivers' preview of the end of a passing zone, to facilitate older drivers' decisions and responses in situations where safe operations dictate that they should abort a passing maneuver. Finally, a recommendation to implement passing/overtaking lanes may be justified in terms of overall system safety and efficiency.

IV. CONSTRUCTION/WORK ZONES

The following discussion presents the rationale and supporting evidence for *Handbook* recommendations pertaining to these five design elements (A–E):

A. Advance Signing for Lane Closure(s)
B. Variable (Changeable) Message Signing Practices
C. Channelization Practices
D. Delineation of Crossovers/Alternate Travel Paths
E. Temporary Pavement Markings

A. Design Element: Advance Signing for Lane Closure(s)

Table 31. Cross-references of related entries for advance signing for lane closure(s).

Applications in Standard Reference Manuals
MUTCD Revision 3, Part VI (1993)
Pg. 3, Item 2b Pgs. 7-8, Sect. on *Advance Warning Area* Pg. 31, Para(s). 5-6 Pg. 40, Para(s). 4-7 Pg. 41, Para(s). 1-2 Pg. 44, Sect. on *Lane Closed Sign* Pg. 50, Sect. on *Other Warning Signs* Pg. 52, Fig. VI-8a, Sign W4-2 Pg. 54, Fig. VI-8c, Signs W9-1 & W9-2 Pgs. 88-90, Sect. 6G-7 Pg. 98, Para. 6 Pg. 99, Items (1) & (2) Pgs. 126-131, 142-143, 148-161, 166-175, 180-185, 190-191, & 194-195, Sect(s). on different type *Lane Closures*

The requirements for safely negotiating a lane closure are an awareness of a decrease in pavement width ahead, and of the direction of the lateral shift in the travel path; a detection of traffic control devices marking the location of the lane drop (beginning of taper); a timely decision about the most appropriate maneuver, taking other nearby traffic into account; and smooth vehicle control through maneuver execution. In the vicinity of a lane closure, the longer the information needs supporting these requirements remain unmet for the *least capable* drivers within the traffic stream, the less likely is a smooth transition through the work area for *all* drivers (Goodwin, 1975). The more time that is required for older drivers to prepare and initiate a merging maneuver, the more dense following traffic (including the adjacent lane) is likely to become; this, in turn, will make gap judgments and maneuver decisions at the point of a lane closure more difficult, and will increase the likelihood of erratic vehicle movements resulting in conflicts between motorists.

Relevant alterations in older adults' cognitive-motor processes include: (1) failure to use advance preparatory information (Botwinick, 1965); (2) difficulty in processing stimuli that are spatially incompatible (Rabbitt, 1968); (3) initiation deficit in dealing with increased task complexity (Jordan and Rabbitt, 1977); and (4) inability to regulate performance speed (Rabbitt, 1979; Salthouse, 1979; Salthouse and Somberg, 1982). Stelmach, Goggin, and Garcia-Colera (1987) found that older adults showed disproportionate response slowing when compared with younger subjects, when there was low expectancy for a required movement. When subjects obtained full information about an upcoming response, reaction time (RT) was faster in all age groups. Stelmach et al. (1987) concluded that older drivers may be particularly disadvantaged when they are required to initiate a movement in which there is no opportunity to prepare a response. Preparatory intervals and length of precue viewing times are determining factors in age-related differences in movement preparation and planning (Goggin, Stelmach, and Amrhein, 1989). When preparatory intervals are manipulated such that older adults have longer stimulus exposure and longer intervals between stimuli, they profit from the longer inspection times by performing better and exhibiting less slowness of movement (Eisdorfer, 1975; Goggin et al., 1989). Since older drivers benefit from longer exposure to stimuli, Winter (1985) proposed that signs should be spaced farther apart to allow drivers enough time to view information and decide which action to take. Increased viewing time will reduce response uncertainty and decrease older drivers' RT.

In focus group discussions consisting of 81 drivers ages 65–86, pavement width transitions were identified as sources of difficulty by 50 percent of the participants (Staplin, Harkey, Lococo, and Tarawneh, 1997). The drivers participating in these discussions suggested longer merging areas to give them more opportunity to find a safe gap and the use of multiple warning signs to allow them to plan their maneuver at an earlier point upstream. Use of multiple signs to give advance notice of downstream work zones and of required maneuvers was also offered as a desired change by older drivers participating in an earlier focus group (Staplin, Lococo, and Sim, 1990).

Lyles (1981) conducted studies on two-lane rural roads to evaluate the effectiveness of alternate signing sequences for providing warning to motorists of construction and maintenance activities that required a lane closure. The signs tested included a standard MUTCD warning sequence, the same sequence augmented with continuously flashing warning lights on the signs, and a sequence of symbol signs (WORKER and RIGHT LANE CLOSED). The most effective sign sequence was one that was flasher augmented; this treatment was twice as effective as similar signs with no warning lights in slowing vehicles in the vicinity of the lane closure.

The use of verbal (text) signing in highway work areas raises sign legibility issues for older drivers. In research conducted to improve the legibility of the RIGHT/LEFT LANE CLOSED and ROAD CONSTRUCTION series signs using test subjects in three age groups (18–44, 45–64, and 65 and older), Kuemmel (1992) concluded the following: (1) signs that increased both letter size and stroke width (SW) while maintaining or increasing the standard alphabet letter series resulted in the best improvement; (2) increasing letter size while decreasing the alphabet series (e.g., from C to B) reduces sign legibility, particularly at night; (3) the use of letter series E, with its 21-percent increase in SW-to-letter height over 200-mm (8-in) series C letters, appears to overcome the problems of irradiation (or overglow phenomenon) with high-intensity retroreflective materials, thus increasing night legibility; (4) the legibility distance of the ROAD CONSTRUCTION signs can be increased by changing the word "construction" to "work," and increasing the letter size from 175-mm series C to 200-mm series C (7-in C to 8-in C); and (5) for the RIGHT LANE CLOSED series, use of symbol signs will have to supplement word legend signs, and for the CENTER LANE CLOSED series, redundancy of sign placement will have to be employed if a 1,200-mm (48-in) maximum sign size is to be maintained. The author pointed out that the minimum required visibility distance (MRVD) for both signs is 101 m at 88 km/h, and 112 m at 104 km/h (331 ft at 55 mi/h and 369 ft at 65 mi/h). The legibility distances obtained in this study for the current standard construction work zone signs ranged from 198 m (650 ft) for the best observers to 43 m (140 ft) for the worst observers. In addition, 85th percentile values were closer to the minimum legibility distances than they were to the maximum legibility distances. This finding reinforces the need for redundant signing during the approach to a work zone.

Finally, a number of studies performed to determine the effectiveness and motorist comprehension of static signs and variable message signs (VMS's)—also referred to as changeable message signs (CMS's)—for lane closures have been reported. A general indication of the importance of VMSs to accomplish lane control in advance of work zones is provided by a field study on a four-lane section of I-35 in San Antonio conducted by Dudek, Richards, and Faulkner (1981) to evaluate the effects of VMS messages on lane changes at a work-zone lane closure. The measure of effectiveness used to evaluate the VMS was the percentage of vehicles that remained in the closed (median) lane as traffic progressed toward the cone taper. The results indicated that the VMS did encourage drivers to vacate or avoid the closed lane, compared with driver responses at the same site without use of the VMS. The percent volumes in the closed lane were significantly lower when a lane-closure message was displayed than during periods when the sign was blank. Specifically, there was a 46 percent greater reduction in the lane volume attributable to the VMS.

During the conduct of field studies for NCHRP project 3-21(2), the relative proportions of traffic in the through and closed lanes approaching construction lane closures were observed for a sample of more than 196,500 vehicles (Transportation Research Board, 1981). Data gathered in Georgia, Colorado, and California were used to compare these lane distributions between baseline (no VMS) conditions and various VMS applications. A fourth data set, gathered in South Carolina, was used to determine relative effects between certain VMS message alternatives (i.e., speed and closure, speed and merge, closure and merge advisories), and various placement configurations (i.e., one VMS at 610 m [2,000 ft] in advance; or one VMS at 1,207-m [3,960-ft] advance placement; or two VMS devices, one at each advance location; or one VMS placed at 1,207 m [3,960 ft] in advance of the taper and an additional arrow panel at the 610-m [2,000-ft] location). Findings indicated increased preparatory lane change activity, smoother lane-change profiles, and significantly fewer "late exits" (exit from a closed lane within 30.5 m [100 ft] of closure) in locations where a VMS was applied at the 1,207-m (3,960-ft) advance location and an arrow panel at the 610-m (2,000-ft) location.

Additional studies of flashing arrow panels at construction sites have shown that they are effective in shifting approaching traffic out of a closed lane (Bates, 1974; Shah and Ray, 1976; Graham, Migletz, and Glennon, 1978; Bryden, 1979; Faulkner and Dudek, 1981). These studies found that arrow panels were effective because they promote early merging into the open lane and fewer vehicles remained in the closed lane at the start of the lane-closure taper. A basis thus exists to assert that a VMS used to give advance notice of the need to exit a lane, followed by the application of an arrow panel, would be of clear benefit to drivers with diminished capabilities resulting from aging, inattentiveness, or transient impairment (e.g., due to fatigue, alcohol, or drugs). While the specific location of the arrow panel in this approach should be consistent with the signing sequence indicated in the MUTCD Part VI, for divided highways, placement of the beginning of the taper is suggested by the findings reported above.

Also during the conduct of NCHRP project 3-21(2), a questionnaire was completed by 489 subjects ranging in age from under 20 to 80 to gather measures of driver detection, recognition, and comprehension of the VMS devices. Twenty percent of the drivers were age 60 and older. Five tested message conditions were: (1) speed and closure advisory (MAX SPEED 45 MPH/RIGHT LANE CLOSED); (2) speed and merge advisory (MAX SPEED 45 MPH/MERGE LEFT); (3) merge and closure advisory (RIGHT LANE CLOSED/MERGE LEFT); (4) speed advisory only (SLOW TO 45 MPH); and (5) closure advisory only (RIGHT LANE CLOSED AHEAD). Drivers consistently reported that the speed advisory and lane closure message combination was most helpful, was the easiest to read, best met their information needs, and would be most likely to cause them to change lanes early and reduce speed.

A recent human factors laboratory study was conducted to determine which VMS message alternatives would be most likely to enhance motorists' compliance with lane control messages in work zones (Gish, 1995). The subjects were divided into two age groups consisting of 24 subjects each: the youngest drivers had a mean age of 23.1 years (range = ages 16–33), and the oldest drivers had a mean age of 70.2 years (range = ages 65–84). The results of this study indicated that older drivers were more likely to reduce their speed and change lanes than the younger drivers, and that both older and younger drivers' compliance with lane-change messages was strongly influenced by surrounding vehicles and by the visibility of the lane closures themselves, which exerts a strong influence on message credibility. Other factors, such as traffic density, static displays, and merge arrows (arrow panels), influence driver compliance with VMS messages. To optimize lane-change compliance, Gish (1995) recommended that static displays, merge arrows, and other devices be used in addition to VMS messages. A need to study the long-term effectiveness on nonstandard messages was also indicated, and potential improvements in work-zone safety and operations through the use of condition-responsive (real-time) traffic control systems that provide continuously updated information to motorists (for enhanced credibility) were identified.

B. Design Element: Variable (Changeable) Message Signing Practices

Table 32. Cross-references of related entries for variable (changeable) message signing practices.

Applications in Standard Reference Manuals
MUTCD Revision 3, Part VI (1993)
Pgs. 57-62, Sect(s). on Portable Changeable Message Signs and *Arrow Displays* Pg. 132, Items 4-5 Pg. 140, Item 7 Pgs. 171, 173, 175 & 177, Fig(s). TA-32–TA-35 Pg. 176, Item 7 Pg. 180, Item 2 Pgs. 181, 183 & 185, Fig(s). TA-37–TA-39 Pg. 182, Item 6 Pg. 191, Fig. TA-42 Pg. 195, Fig. TA-44

The effectiveness of variable message signs (VMS's), gauged in terms of observable driver behaviors that traffic management procedures are designed to elicit, rests upon a set of reasonably well-understood human factors. A motorist information system must be rational, relevant, and reliable. Driver sensory/perceptual and cognitive capabilities must be thoughtfully considered to ensure that a message will be acquired and then understood, recalled, and applied by the driver within a desired timeframe; the message must seem to clearly apply to the driver *and* to reflect current conditions to be credible; and it must be accurate in describing what the driver experiences downstream. The credibility of a highway advisory message certainly depends in part upon a presentation strategy that is "rational," but it also must be perceived to be relevant to the individual motorist, and reliable to the point of being virtually error-free. Reliability requirements—being dependent on real-time data on operations as input to the traffic control system—are most difficult to meet, but probably the most important if high rates of compliance in drivers' vehicle control decisions are ever to be realized.

A motorist's ability to use highway information is governed by: (1) *information acquisition*, or how well the source can be seen or heard and (2) *information processing*, or the speed and accuracy with which the message content can be understood, and its ease of recall by the motorist after message presentation is completed.

In the acquisition of VMS information, a visual task, the key factors are: (1) its *conspicuity*, or "attention-getting value" to the motorist; (2) the size, brightness (contrast), stroke width-to-height ratio, and spacing of individual characters of text, which together determine the *legibility* of the message; (3) the *placement* of the VMS device—overhead versus one side versus both sides of the highway—which affects its likelihood of being blocked from a motorist's view by other vehicles, as well as the "eyes away from the road" time required to fixate upon the message; and (4) the *exposure time*, or available viewing time, of each message phase presented on a VMS.

Conspicuity is generally not a problem for any type of VMS under low traffic volumes, although under high volumes with a significant mix of heavy vehicles, a motorist may fail to notice a roadside device because of obscuration. Good conspicuity is achieved by overhead devices under all conditions. The attention value of a VMS display can be maximized by flashing operations, but this also works against information acquisition by reducing exposure time and legibility; this strategy is thus uniformly discouraged for an entire message. In rare circumstances, for a unit of information deemed particularly critical by the highway authority, the flashing of a single text element *within* a message at a slow rate may be justified. The use of flashing text may help bring the sign to the attention of an older driver who has a reduction in his/her useful field of view and may otherwise fail to notice the sign. If it is standard policy to leave the signs blank, then the mere display of a message will capture the driver's attention. However, if the VMS in question always has some type of message displayed, then *slowly* flashing (e.g., two cycles per phase) the problem statement line only may be warranted to attract attention. A preferred strategy under such circumstances would be to activate a flashing warning light separate from, though clearly attached to, the VMS.

The legibility of a VMS is influenced by the same factors influencing character and message legibility of static signs, including the key factor of driver visual performance capability. Letter acuity declines during adulthood (Pitts, 1982) and older adults' loss in acuity is accentuated under conditions of low contrast, low luminance, and where there is crowding of visual contours (Sloane, Owsley, Nash, and Helms, 1987; Adams, Wong, Wong, and Gould, 1988). In any event, the legibility for current VMS's is determined primarily by the technology and the device configuration (numbers of rows, characters per row, and number, size, and spacing of pixels per character) as fabricated by a given manufacturer, and for all practical purposes can be treated as a fixed factor—modified by environmental considerations—in considering whether a particular system as implemented in the field will meet motorists' needs.

For any given speed, older drivers' needs dictate a legibility distance that permits the entire VMS message to be read *twice in its entirety*. As a general rule, at least 305 m (1,000 ft) of legibility distance for a motorist with 20/40 visual acuity should be provided on a 88-km/h (55-mi/h) facility. Of the studies that assessed various character matrix forms (number of elements per character cell), most found a 7 x 9 element matrix to be necessary when using lowercase letters, because of the descenders and ascenders, but a 5 x 7 font was generally deemed acceptable with uppercase-only lettering. The MUTCD specifies a minimum legibility requirement of 198 m (650 ft) under both day and night conditions for Portable Variable Message Signs. Given that the most common format for a portable sign is 450-mm (18-in) tall characters arranged in three lines of eight characters, this provides for a legibility distance of 0.44 m/mm (36 ft/in) of letter height. Thus, letter sizes of at least 450 mm (18 in) should be used to accommodate older drivers' diminished visual acuity. Other variables found to significantly effect VMS legibility for older observers are font, letter width-to-height ratio, contrast orientation, letter height, case, and stroke width (Jenkins, 1991; Mace, Garvey, and Heckard, 1994). The most consistent finding across studies evaluating VMS design elements was that the results found for older drivers were quantitatively but not qualitatively different from those of their younger counterparts. That is, if a manipulation of a variable resulted in an improved score for younger observers, it almost invariably improved older observer performance.

The "target value," legibility, and viewing comfort of light-emitting diodes (LED's) and fiber-optic VMS technologies were compared with flip-disk and conventional overhead guide signs in a field study conducted by Upchurch, Baaj, Armstrong, and Thomas (1991). Younger (ages 18–31) and older (ages 60–79) subjects in this study demonstrated mean daytime target values for fiber-optic, LED, and flip-disk technologies that all were significantly better (longer) than the values for conventional overhead signs. Under nighttime conditions, however, the poorest performance (shortest distances) were demonstrated by both age groups for the flip-disk technology, falling below the conventional sign values as well. The fiber-optic and LED signs again exceeded the conventional signs, based on nighttime mean target value, with the fiber-optic technology showing a slight superiority for older drivers. Under backlight (sun behind sign) and washout (sun behind driver) conditions, target values for all sign types decreased substantially and the differences among sign types diminished, but the fiber-optic technology still resulted in the best overall performance, across age groups.

Legibility distance results tended to favor the conventional signs, followed by the fiber-optic signs, LED signs, and flip-disk technology. Mean daytime legibility distances for each sign type in this study were as follows: fiber-optic—0.74 m/mm (61 ft/in); LED—0.51 m/mm (42 ft/in); flip-disk—0.47 m/mm (39 ft/in); and conventional—1.07 m/mm (88 ft/in). Under nighttime conditions, the conventional signs again could be read at the longest mean distances, followed closely by the fiber-optic and LED signs, with the flip-disk technology showing the poorest performance. Backlight conditions favored the fiber-optic technology, and washout conditions favored the conventional signs; in both cases, however, the flip-disk technology resulted in the shortest legibility distances. Using a threshold for minimal acceptable legibility distance of 191 m (628 ft), the study concluded that flip-disk signs are deficient under all conditions *except* midday daytime viewing, LED signs are deficient under both backlight and washout sun conditions, and fiber-optic signs are deficient only with the sun glare present under backlight conditions.

Mean discomfort ratings were consistent with these patterns of results. Fiber-optic and conventional signs were assigned the best (lowest discomfort) ratings under daytime conditions, by younger and older drivers alike. LED signs caused slightly more discomfort for older subjects, and flip-disk signs resulted in the highest discomfort ratings, especially for older drivers. Under nighttime conditions, only the flip-disk technology resulted in high discomfort ratings. Discomfort ratings were more even, and much higher, across sign types under backlight conditions where the sun was behind the sign, though flip-disk signs still were rated the worst by both age groups. Under washout conditions, subjects reported little discomfort for either the fiber-optic or conventional signs, but much greater and roughly equivalent levels of discomfort with the LED and flip-disk technologies.

Table 33 contains legibility distances from the Upchurch et al. (1991) study. For older drivers, the legibility distances are lower due to the well-documented degradation of visual performance with age. Unfortunately, this is the only study that has assessed legibility distances for older observers. The legibility distances for conventional bulb matrix and LED/flip-disk hybrid VMS's were estimated from the results of the Upchurch data and data cited in Dudek (1991).

Table 33. Day and night predicted legibility distances (ft) for various sign technologies.

Sign Technology (Character Height, in)	Daytime Legibility Distances (ft)		Nighttime Legibility Distances (ft)	
	Younger Observers	Older Observers	Younger Observers	Older Observers
Fiber-optic (16 in)	1006	959	687	667
Light-emitting diodes (17.8 in) *	812	681	794	602
Flip-disk (18 in)	731	667	363	348
Bulb matrix (18 in)	800	671	750	569
Hybrid LED/flip-disk (18 in)	731	667	794	602

* Legibility distance of this technology decreases over time, because as LED's age, they become less bright.

1 ft = 0.305 m
1 in = 25 mm

The older driver legibility distances in table 33 should be assumed to represent the legibility distances for the various types of technology represented. This ensures that the needs of older drivers have been met. The results suggest that flip-disk VMS's should not be used at night along roadways where average speeds reach or exceed about 88 km/h (55 mi/h).

Although the bulb matrix VMS was assessed by Upchurch et al. (1991), no legibility distances for that sign were reported. Legibility distances for this type of VMS have been obtained; however, it is unknown whether any older observers have been used in assessing legibility distances. Dudek (1991) cited a study in which bulb matrix VMS's provided legibility distances of 244 m (800 ft) during the day and 229 m (750 ft) at night. These distances are similar to the legibility distances obtained by Upchurch et al. (1991) for LED-type VMS's using younger observers. Until psychophysical data can be obtained for older observers viewing bulb matrix signs, the legibility distances for older observers are assumed to be roughly 204 m (671 ft) during the day and 173 m (569 ft) at night. These estimates are based on applying the ratio of older-to-younger legibility distances for the LED-type display.

There are also a number of hybrid VMS's that were not included in the Upchurch et al. study. Hybrid VMS's apply various combinations of sign technologies listed in table 33 within a single sign. Product literature for one manufacturer's hybrid LED/flip-disk sign states that the sign provides 274 m (900 ft) of legibility distance during the day and greater than 274 m (900 ft) at night, using character heights of 450 mm (18 in). Unfortunately, the methods used to obtain these legibility distances are unknown. Since the sign uses the reflective flip-disk technology during daytime and the LED's at night, the legibility distances for older observers for the daytime flip-disk in table 33 (203 m [667 ft]) should be used as a more realistic estimate of legibility distance with LED/flip-disk hybrids. For nighttime viewing, use the nighttime LED legibility distance (183 m [602 feet]) in table 33.

VMS placement affects information acquisition under heavy traffic conditions where a center lane driver's view of a roadside device may be obscured for lengthy intervals. If a facility has more than two lanes, a consideration may be given to placement of a portable VMS in the median—space permitting and where glare from opposing vehicles is absent or minimal due to a large glare angle—rather than on the right shoulder, since lane control practices for heavy trucks are common throughout many corridors.

A motorist's reading time for a VMS message dictates the required exposure time at a given speed. Exposure time is the length of time a driver is within the legibility distance of the message. The minimum recommended exposure time per page (phase) for a three-line VMS is 3 s, aside from a consideration of any particular set of driver characteristics. However, while some jurisdictions have selected briefer exposure times, the increasing numbers of older drivers on limited-access highways makes an even stronger case for the 3-s minimum per page. Reading time is the time it actually takes a driver to read a sign message. In instrumented vehicle studies conducted in light traffic with familiar drivers on a rural freeway, reading times averaged 1 to 1.5 s per unit of information (Mast and Ballas, 1976). Reading times under "loaded" driving conditions would be higher, such as under extreme geometry, heavy traffic volumes, large volume of truck traffic, traffic conflicts, or poor climatological conditions. More recent field research using unfamiliar drivers has indicated that a minimum exposure time of 1 s per short word (four to eight characters) or 2 s per unit of information, whichever is larger, should be used (Carvell, Turner, and Dudek, 1978; Messer, Stockton, and Mounce, 1978; Weaver, Richards, Hatcher, and Dudek, 1978; Dudek, Huchingson, Williams, and Koppa, 1981). A unit of information is a data item given in a message, that can answer one of the following questions: (1) what happened? (2) where? (3) what is the effect on traffic? (4) for whom is the advisory intended? and (5) what driver action is advised? Thus, the exposure time for a three-line message could vary from 3 s to as much as 6 s, with each phase of a portable VMS at the lower end of this range and with each permanent VMS phase (page) at the upper end, due to differences in the number of characters per line. Reducing the exposure time per phase is warranted only when information is being repeated. For example, a three-line message may be displayed for only 2.5 s if it is a second phase of a two-phase message which repeats one or two lines from the first phase. If the second phase presents new information, the recommended *minimum* exposure time for both phases remains 3 s.

For a given operating speed, exposure will increase with increasing legibility distance. For example, an overhead sign message legible at 198 m (650 ft) will be exposed to drivers traveling at 88 km/h (55 mi/h) for approximately 8 s. With a legibility distance of 305 m (1,000 ft), the message will be exposed for about 12 s. Legibility distances for portable VMS's vary from the minimum of 198 m (650 ft) specified by the MUTCD Part VI and American Traffic Safety Services Association (ATSSA) to over 305 m (1,000 ft), depending on the technology. Permanent VMS's generally have legibility distances in the higher range of 274–366 m (900–1,200 ft). However, there is a point at which a sign becomes unreadable during a driver's approach to a VMS, which reduces the legibility distance, particularly for side-mounted VMS's. This unreadable distance, which is dependent on the number of lanes and the sign technology, as well as how far the sign is placed from the roadway edge or how high above the roadway it is mounted, ranges from 85 m to 128 m (280 ft to 420 ft). In an existing system, therefore, required exposure times dictate the maximum length of message that can be displayed, and *in* *all cases, it is desirable that motorists be able to read the entire message on an (unobstructed) VMS twice.*

The calculated maximum exposure duration of a message should not exceed 9 s. For two-phase messages, a separate requirement is needed to meet the needs of drivers. In this case, 3 s is added to the required exposure time because of the asynchrony between the time the driver can read the VMS and the onset of VMS phase displayed. In other words, the phase that the driver reads initially may have already been displayed for 2 s by the time he or she can read it. Thus, the driver will not have enough time to read this phase and will need to view that phase again. The net result is that 3 s needs to be added to the required exposure time to allow drivers to read the phase that first came into view a second time. Since the maximum recommended exposure time is 9 s, only 6 s of actual message reading time is allowed on a two-phase VMS, whereas the full 9 s can be used for a single-phase message. The important point here is that single-phase messages can more efficiently convey information to drivers. When use of a single-phase VMS is not possible because of message length, multiple devices with a single phase on each device will be superior with respect to drivers' limitations for message acquisition versus multiple phases on a single device. Part VI of the MUTCD (para. 6F-2) specifies that when multiple VMS's are used, they shall be placed on the same side of the roadway, separated by at least 305 m (1,000 ft).

For these reasons, the *maximum* number of phases used to display a message on a permanent VMS should be two. The most effective format for VMS message presentation is a single phase which consists of a maximum of three units of information, but if two are required, each should be worded so that it can stand alone and still be understood. Portable VMS devices, though limited to fewer characters per line, should also be restricted to two phases. At high speeds (88 km/h [55 mi/h]), a driver may only have 2.8 to 4.6 s to read a message on a side-mounted VMS, depending on the available legibility distance. For this reason, messages should be restricted to one phase at high speeds.

The motorist's need for rapid understanding and integration of message components also focuses attention on the formatting of multiword text displays. The main concern is with "units of information"—i.e., where and how to divide phrases—and with the use of abbreviations and contractions in VMS messages.

Work zones constitute driving situations that require a high amount of controlled processing, and data show that cognitive ability scores that measure processing efficiency decline with age (Ackerman, 1987). In fact, sensory memory, working memory, and divided attention all show a decline with aging and must be considered in the display of messages on VMS's. Sensory memory is a high-capacity, briefly accessible register from which information is lost through decay or interference. While there is evidence that older adults require slightly longer to establish a legible "icon" in sensory memory, another set of findings suggests that with advancing age, images are instead more susceptible to masking by other (successive) stimuli (Walsh, Till, and Williams, 1978; Cerella, Poon, and Fozard, 1982). This suggests that a message should be limited to a single phase, or certainty no more than two, because multiple phases will interfere with message comprehension. There is also considerable evidence that older adults have poorer working memory function than younger adults (Salthouse, 1991; Salthouse and Babcock, 1991). This suggests that message length be limited to the fewest, most relevant units possible. Finally, older adults are particularly disadvantaged when they are required to use working memory to manage multiple tasks (Ponds, Brouwer, and van Wolffelaar, 1988). Van Wolffelaar, Brouwer, and Rothengatter (1990) found that older drivers made more tracking (steering) errors when required to attend and respond to a dot-counting task and a task that required them to monitor peripheral events. In this study, older adults also showed a dramatic increase in the rate of nonresponding on the dot-counting task under multiple task conditions, compared to younger subjects. Van Wolffelaar et al. (1990) concluded that there is a disproportionately greater problem for older adults in divided attention situations and directly linked this to a higher accident rate for older adults in time-pressured, complex traffic situations.

The minimum required information for traffic management includes: (1) a statement of the problem and (2) the action statement(s)—i.e., a driver needs to know what to do and one good reason for doing it. Additional elements are included as needed for a specific situation. The key here is not to burden the driver with unnecessary information. Only about two-thirds of drivers are able to recall completely four pieces of information (problem, effect, attention, and action); however, 80–90 percent can recall the action message (Huchingson, Koppa, and Dudek, 1978). Two problems in message presentation must be avoided: (1) providing too much information in too short a time and (2) providing ambiguous information that leaves either the intent of the message or the desired driver response uncertain.

The first problem does not refer solely to reading time difficulties, as discussed above; instead, it refers to the number of ideas, or "information units," contained in a message. Certainly, the number of words displayed on a sign is important, but so is the manner in which words are grouped. Units containing one word (DELAY), two words (DELAY AHEAD), or many words (MAJOR DELAY AT HIGH STREET) are equally difficult to remember when the display is no longer in sight. However, a series of, say, six units of information in a message displayed on a permanent VMS will be easier to remember if presented in two phases of three units each than if all six units are presented on a single phase. Studies have concluded that no more than three units of information should be displayed on one sequence when all three units must be recalled by drivers (Huchingson et al., 1978; Dudek et al., 1981; Gish, 1995).

Gish (1995) conducted a human factors laboratory study addressing the perceived timeliness, accuracy, and credibility of VMS messages using both younger (ages 16–33) and older (ages 65–84) test subjects. Results showed that correct recall of the first VMS phase (a downstream speed advisory) was nearly 100 percent for both age groups. However, successive phases of information (containing downstream delay and route diversion information) were recalled less accurately. For the delay information (second phase), correct recall for the younger subjects was about 82 percent, versus 60 percent for the older subjects. For route numbers (third phase), correct recall was 55 percent for the younger subjects and 19 percent for older subjects. These results reinforce the earlier recommendation that a maximum of two phases should be used.

When a message *must* be divided into two phases, it is desirable to repeat key words from the first phase on the second phase, to provide assurance that all drivers see the message at least once. This also allows information rehearsal, as provided by an additional "learning trial," which will facilitate message recall when the device is no longer in sight. A recommended standard practice is therefore to put the problem on line 1, the location on line 2, and alternate either the effect and action *or* diversion information on line three, repeating lines 1 and 2 on both phases.

The second type of problem can occur when an unfamiliar word or abbreviation is used, when a word is hyphenated or a phrase is divided inappropriately, or when an abbreviation or a word can mean different things in different word pairings or contexts. Ambiguity occurs, for example, when CENTER LANE is used on a freeway with four or more lanes in one direction. Another example is the use of LANE CLOSED versus LANE BLOCKED, to denote a prolonged closure for construction/maintenance versus a temporary blockage due to an accident or stall. To foster the most simple and consistent practice for motorists, LANE CLOSED is recommended under both roadwork and incident conditions, because at the time the message is displayed, the lane is effectively closed. Finally, neither FREEWAY BLOCKED nor FREEWAY CLOSED should ever be used when at least one lane is open to traffic.

Abbreviations also have the potential to be misunderstood by some percentage of drivers, exacerbating message comprehension problems for individuals with (age-related) diminished capabilities. It has been determined that certain abbreviations are understood by at least 85 percent of the driving public independent of the specific context (e.g., BLVD = boulevard). A second category of abbreviations are understood by at least 75 percent of the driving population but *only* with a prompt word, (e.g., LOC means "local" when shown with "traffic"). Other abbreviations are prone to be frequently confused with another word (e.g., WRNG could mean either "warning" or "wrong") and should be avoided. Following are lists of abbreviations in three categories, extracted from Dudek et al. (1981): (1) those that are acceptable (understood by at least 85 percent of the driving population) when shown alone (table 34); (2) those that are *not* acceptable and, therefore, should *not* be used (table 35); and (3) those that require a prompt word (table 36). Table 34 also includes abbreviations taken from the MUTCD, as well as common contractions used in the English language.

Table 34. "Acceptable" abbreviations for frequently used words.
Source: Dudek, Huchingson, Williams, and Koppa (1981).

Word	Abbreviation
Alternate	ALT
Avenue	AVE
Boulevard	BLVD
Can Not	CAN'T
Center	CNTR
Do Not	DON'T
Emergency	EMER
Entrance, Enter	ENT
Expressway	EXPWY
Freeway	FRWY, FWY
Highway	HWY
Information	INFO
It Is	IT'S
Junction	JCT
Left	LFT
Maintenance	MAINT
Normal	NORM
Parking	PKING
Road	RD
Service	SERV
Shoulder	SHLDR
Slippery	SLIP
Speed	SPD
Street	ST
Traffic	TRAF
Travelers	TRVLRS
Warning	WARN
Will Not	WON'T

Table 35. Abbreviations that are "not acceptable."
Source: Dudek, Huchingson, Williams, and Koppa (1981).

Abbreviation	Intended Word	Common Misinterpretation
ACC	Accident	Access (Road)
CLRS	Clears	Colors
DLY	Delay	Daily
FDR	Feeder	Federal
L	Left	Lane (Merge)
LT	Light (Traffic)	Left
PARK	Parking	Park
POLL	Pollution (Index)	Poll
RED	Reduce	Red
STAD	Stadium	Standard
WRNG	Warning	Wrong

Table 36. Abbreviations+ that are "acceptable with a prompt."
Source: Dudek, Huchingson, Williams, and Koppa (1981).

Word	Abbreviation	Prompt
Access	ACCS	Road
Ahead	AHD	Fog*
Blocked	BLKD	Lane*
Bridge	BRDG	[Name]*
Condition	**COND**	**Traffic***
Congested	**CONG**	**Traffic***
Construction	CONST	Ahead
Downtown	**DWNTN**	**Traffic***
Eastbound	**E-BND**	**Traffic**
Exit	EX, EXT	Next*
Express	EXP	Lane
Frontage	**FRNTG**	**Road**
Hazardous	HAZ	Driving
Interstate	I	[Number]
Local	**LOC**	**Traffic**
Major	MAJ	Accident
Mile	MI	[Number]*
Minor	MNR	Accident
Minute(s)	MIN	[Number]*
Northbound	**N-BND**	**Traffic**
Oversized	OVRSZ	Load
Prepare	PREP	To Stop
Pavement	PVMT	Wet*
Quality	QLTY	Air*
Roadwork	**RDWK**	**Ahead [Distance]**
Route	RT	Best*
Southbound	**S-BND**	**Traffic**
Temporary	**TEMP**	**Route**
Township	**TWNSHP**	**Limits**
Turnpike	TRNPK	[Name]*
Upper, Lower	UPR, LWR	Level
Vehicle	VEH	Stalled*
Westbound	**W-BND**	**Traffic**
Cardinal Directions	N, E, S, W	[Number]

* Prompt word should precede abbreviation.

\+ The words and abbreviations shown in normal type are understood by at least 85 percent of the driving population. Those shown in boldface type are understood by at least 75 percent of the driving population, and public education is recommended prior to their usage.

C. Design Element: Channelization Practices

Table 37. Cross-references of related entries for channelization practices.

Applications in Standard Reference Manuals	
MUTCD Revision 3, Part VI (1993)	
Pg. 5, Item 5b	Pgs. 63-71, Sect. 6F-5
Pg. 84, Items (2) & (3)	Pg. 89, Para. 1
Pg. 93, Para. 2	Pg. 94, Para. 2
Pg. 95, Para. 5	Pg. 97, Para. 1
Pgs. 100-101, Item (4)	Pg. 112, Item 3
Pg. 113, Fig. TA-3	Pgs. 117 & 119, Fig(s). TA-5–TA-7
Pg. 126, Item 3	Pgs. 127, 129 & 131, Fig(s). TA-10–TA-12
Pg. 136, Item 2	Pgs. 137 & 139, Fig(s). TA-15 & TA-16
Pgs. 143 & 145, Fig(s). TA-18 & TA-19	Odd No. Pgs. 149-175, Fig(s). TA-21–TA-34
Pg. 150, Item 3	Pg. 158, Item 2
Pg. 162, Item 4	Pg. 164, Item 4
Pg. 170, Item 4	Pg. 174, Item 5
Pg. 178, Item 6	Odd No. Pgs. 179-195, Fig(s). TA-36–TA-44
Pg. 182, Item 3	Pg. 184, Item 3
Pg. 188, Item 3	Pg. 190, Item 3

Channelization systems include the use of cones, posts, tubes, barricades, panels, drums, amber-flashing and steady-burn lights, and standard and raised/recessed pavement markings. They are used to direct motorists into the open lanes and to guide them through the work area. They must provide a long detection distance and be highly conspicuous under both day and night conditions. Using data collected by the police, it has been estimated that anywhere from 80 to 86 percent of the accidents in work zones can be attributed to driver error (Nemeth and Migletz, 1978; Hargroves and Martin, 1980). Hargroves and Martin (1980) found that accidents with fixed objects within a work-zone account for a greater percentage than other accident types, such as rear-end or sideswipe. Nemeth and Migletz (1978) found that nighttime accidents are concentrated in the taper area. The most significant problems with channelization in work zones have been identified as: (1) failure to use, or hazardous use of, temporary concrete barriers; and (2) inadequate or inconsistent use of devices and methods in closing roadways and establishing lane-closure tapers (Humphreys, Maulden, and Sullivan, 1979).

Older drivers, like alcohol-impaired and fatigued drivers, show reduced sensitivity to contrast. Olson (1988) pointed out that the brightness of a traffic control device is the main factor in its attention-getting capability: in a visually complex environment, the brightness must be increased by a factor of 10 to achieve conspicuity equivalent to that found in a low-complexity environment. A major problem at night is reduction in contrast sensitivity, which makes it difficult to see even large objects when they cannot be distinguished from their background. Older drivers also have difficulty processing information due to less effective scanning behavior and eye movements, diminished visual field size, difficulty in selective attention, and slower decisionmaking. Inconsistent use of barrels and traffic cones to delineate the travel path may be a particular problem for older drivers, especially when applied in the presence of remnants of old lane markings, because such inconsistency is confusing and older drivers (and inattentive drivers) are not able to react as quickly to conflicting traffic cues (National Transportation Safety Board, 1992). To compensate for their slower information-processing capabilities, their reduced visual capabilities, and their slower reaction time, older drivers often drive more slowly. Although driver age was not studied, Hargroves and Martin (1980) found that slow-moving vehicles were overrepresented in work-zone accidents. Older drivers also show reductions in lane-keeping ability, which is further compromised when they are required to attend to other tasks, in unfamiliar surroundings. Finally, steering abilities may be adversely affected by physical problems such as arthritis.

McGee and Knapp (1979) performed an analytic study to develop a performance requirement/standard for the detection and recognition of retroreflective devices (cones, drums, panels, and barricades) used in work zones. The performance standard developed in this study, presented in terms of visibility requirements (i.e., the distance at which motorists should be able to detect and recognize the devices at night) and established using the principles of driver information needs and the requirement for decision sight distance, calls for a minimum visibility distance of 275 m (900 ft) when illuminated by the low beams of standard automobile headlights at night under normal atmospheric conditions.

Pain, McGee, and Knapp (1981) evaluated the effectiveness of traffic cones and tubes, vertical panels, drums, barricades, and steady-burn lights in laboratory studies, in controlled field studies, and at actual construction sites. Overall, there were no major differences between the device categories in the daytime. At night, barricades, panels, drums, cones, and tubes were also equivalent when the optimized cone and tube reflectorization was used. Posts and cones with 150 mm (6 in) of collar did not elicit an equivalent level of driver behavior, especially at night. Interestingly, in comparing the meaning of chevrons versus stripes, it was found that diagonal, horizontal, and vertical stripes conveyed no consistent directional information; chevrons, though less easily detected than the stripe patterns, effectively and unambiguously indicated that a movement to the left or right was required. Since diagonal, horizontal, and vertical stripes conveyed no consistent direction information, Pain et al. (1981) concluded that there was no reason to have a diagonal stripe pattern for left and right "sidedness." They pointed out, however, that only one direction of diagonal should be allowed in an array so there is always a consistent pattern or image on devices.

In terms of device spacing, comparisons of regular speed-limit spacing (16.8 m [55 ft] in the test), half spacing (8.4 m [27.5 ft]), and double spacing (33.5 m [110 ft]) of Type I barricades and 200-mm x 600-mm (8-in x 24-in) panels showed that changes in spacing produced little impact on driver behavior. There were no significant speed or lateral placement differences between half, regular, and double speed-limit spacing during the day. At night, however, when devices were placed at half spacing, they produced a speed reduction, apparently from the illusion that the motorist was going faster than he or she actually was. Devices placed at double spacing tended not to perform as well as when they were placed at regular speed-limit spacing, as drivers made lane changes and detected arrays of traffic control devices sooner with shorter spacings. From these findings, Pain et al. (1981) recommended that: (1) all devices be placed at speed limit spacing for most conditions and, in all cases, along the taper or transition section; (2) if there is no construction work or hazard in the closed lane for a substantial length, or traffic delays, the spacing can be increased to no more than twice the speed limit; and (3) shorter spacings may prove to be useful where speed reduction is desired.

Device-specific findings by Pain et al. (1981) are as follows:

- *Traffic cones*. (1) They perform as well as other devices during daytime, with long detection distance and adequate lane-change distances. (2) Bigger is better: 900-mm (36-in) cones are more effective than 700-mm (28-in) cones; 700-mm (28-in) cones are better than 450-mm (18-in) cones. (3) At night, 3,750–5,000 mm^2 (150–200 in^2), or roughly the amount in a 300–350-mm (12–14-in) collar of highly reflective material (with specific intensity per unit area [SIA] of at least 250), is needed for effectiveness. Even higher brightness materials (e.g., polycarbonate) enhance driver response characteristics and are preferable.

- *Tubes (tubular cones)*. (1) During daytime, 700-mm and 1,050-mm (28-in and 42-in) tubes are as effective as cones, but 450-mm (18-in) tubes are ineffective and not recommended for lane closures or diversions on high-speed facilities. (2) At night, tubes with at least a 300-mm (12-in) highly reflective band are equally as effective as cones.

- *Vertical panels*. (1) Laboratory results showed that compared with the barricade, the vertical panel is more easily detectable. (2) Vertical panels are equally as effective (detectable) as Type I barricades, and vertical panels promote earlier lane changing than barricades. (3) The minimum width dimensions of the panel should be 300 mm (12 in) rather than 200 mm (8 in), especially when used at night and on high-speed facilities.

- *Drums*. (1) Drums are highly visible and detectable from long distances, during both day and night. (2) Drums promote lane changing further upstream of the taper than other devices. (3) Drums are associated with a speed reduction. (4) Drums are a dangerous object when hit.

- *Barricades*. (1) The Type I barricade is as effective as other devices. (2) The Type II barricade is no more detectable than the Type I barricade. (3) The 300-mm x 900-mm (12-in x 36-in) barricade is more conspicuous than the 200-mm x 600-mm (8-in x 24-in) barricade.

Other findings were reported for comparisons of steady-burn lights and Type II and Type III sheeting. The steady-burn lights provided the longest detection distances at night compared with all other materials, and they more than tripled the distance (or zone) in which lane changing occurred before the taper. In comparisons of Type II sheeting and Type III sheeting on cone and tube optimization tests, Type III was significantly better at night on a flat road. Narrow-angle sheeting, even though offering high brightness, was not effective under certain sight geometry characteristics, such as hills and curves. Type III sheeting and steady-burn lights were comparable in terms of point-of-lane-change and array detection distance; however, the authors noted that the effect of vertical or horizontal curvature must be considered.

There have been mixed results regarding the effectiveness of steady-burn lights in highway work zones. The use of steady-burn lights mounted on channelizing devices has been shown to significantly influence driver behavior in some work-zone configurations, and they are particularly effective in left-lane closures (KLD Associates, 1992). Although older drivers (age

55 and older) consistently showed poorer performance than younger drivers in all study conditions, evidence was found that the use of lights improved the performance of older test subjects. The variables manipulated in this study included work-zone configuration (left-lane, right-lane, and shoulder closures), device type (panels versus drums), and light placement (every device, alternate devices, no lights). Drivers of all ages were able to identify lane and shoulder closures from greater distances when lights were used on channelization devices, as opposed to when the channelizing devices were used alone. Steady-burn lights produced a higher percentage of correct responses (determining the direction the channelizing devices were leading) for all driver age groups when used in left-lane closures than in right-lane closures. Interestingly, the use of lights on *every other* drum or vertical panel (placement on alternate devices) generated more correct responses than the use of lights on consecutive devices. More generally, the literature suggests that in environments characterized by high-speed operations, compromised visibility due to inclement weather, and/or complex maneuvers required as a result of work-zone configuration, the deployment of steady-burn lights should be considered on *all* channelizing devices used for right-lane closures.

However, Pant, Huang, and Krishnamurthy (1992) obtained a different result when they examined the lane-changing behavior of motorists in advance of tapered sections as they drove an instrumented vehicle through work zones during the day, at night when steady-burn lights were placed on drums, and at night when the steady-burn lights were removed. They measured the traffic volume at several locations in each lane in advance of the taper. Results showed that the steady-burn lights had little effect on driver behavior in the work zones studied. It was concluded by Pant et al. that steady-burn lights have little value in work zones that employ drums with high-intensity sheeting *and* a flashing arrow panel as channelizing devices.

Opiela and Knoblauch (1990) conducted laboratory and field studies to determine the optimal spacing and use of devices for channelization purposes in the taper or tangent sections of work zones. In the laboratory study, the recognition distances of eight different device types, spaced at the standard distance and at 1.5 and 2.0 times the standard distance, were measured for 240 subjects. Results indicated variability between the performance of most channelizing devices across the spacings tested. Right- and left-lane closures were then used at six actual work zones, to test the various device spacings under both day and night conditions. Field data were collected at four points equally spaced over 610 m (2,000 ft) before the work zone and the activity at the start of the taper for the lane closure, according to the premise that the most effective treatment would minimize the percentage of traffic in the closed lane at the start of the taper. Statistical analysis of 2,100 5-minute observation periods showed that neither type of device (round barrels, oblong barrels, Type II barricades, and cones with reflective collars) nor device spacings (16.8, 24.4, and 33.5 m [55, 80, and 110 ft]) had a significant effect on driver lane-changing behavior.

Cottrell (1981) also found that driver lane-change response was not strongly dependent on the channelizing device employed in a work-zone taper. The objective of this study was to evaluate the effectiveness of alternative orange-and-white chevron patterns on vertical panels and barricades that form an arrow pointing in the direction in which traffic is being diverted, compared with traffic cones, simulated drum vertical panels, and Type II barricades and vertical panels with standard orange-and-white striping patterns. The measure of effectiveness was the position of lane changing relative to the transition taper. Although the subjective evaluation revealed that chevron patterns were preferred over the presently used patterns because of their clear directional message, the positions of lane changing were similar for the stripes and chevrons. With respect to the spacing of devices, it was generally found that lane changes occurred more frequently at greater distances from the taper when the devices were spaced every 12 m (40 ft), as opposed to every 24.4 m (80 ft).

In a supplemental test, the effectiveness of the Jersey concrete barrier was compared with that of the channelizing devices studied (Cottrell, 1981). The barrier was marked with steady-burn warning lights and 150-mm (6-in) reflectors and had a slope of 16:1 for the 58.5-m (192-ft) taper. The Jersey barrier was rated equal to the cone during the daytime and lower than all other devices based on the lane-change positions. It was recommended that a supplemental taper of channelization devices be used with the Jersey barrier. In a study of concrete barrier visibility, Pain et al. (1981) found that reflectors were superior to reflectorized tape. Logically, the most conspicuous types of reflective devices, such as those containing cube-corner lenses, will be most effective in this regard.

Overall, Pain et al. (1981) concluded that most devices show relatively successful detection and path guidance performance. However, a major deterrent to effectiveness is not the device itself; poor positioning, dirt, and overturned devices destroy the visual line or path created by the channelizing devices. Therefore, *although use of appropriate devices is important, of equal importance is conscientious set-up and care of the work zone.*

In terms of the threat posed to drivers by passenger compartment intrusion or interference with vehicle control, or the threat to workers and other traffic from impact debris, plastic drums, cones, tubes, and vertical panels used as channelizing devices presented no hazards in full-scale vehicle crash tests (Bryden, 1990). However, Types I and II barricades and portable signs and supports formed impact debris, which was often thrown long distances through work zones, posing a threat to workers and other traffic. The American Traffic Safety Services Association (ATSSA) is opposed to the use of metal drums in work zones as channelizing devices, as they pose a hazard to motorists as well as workers in the zone (TranSafety, 1987). They suggest the use of plastic drums, which are safer. Riedel (1986) described studies showing that a substantial number of work-zone accidents occur in the taper and the crossover where channelization devices are located. The frequency of accidents involving drums has led to the use of forgiving devices such as plastic drums, which in tests have been shown to be safer than steel drums. Juergens (1972) noted that because barricades are inherently fixed-object hazards, they should not be used as primary delineation to guide traffic. Further, they should not be used unless the construction hazard the motorist may encounter is greater than the hazard of striking the barricades. A concern with the use of steady-burn lights mounted on channelizing devices was highlighted in full-scale vehicle crash tests evaluating the performance of work-zone traffic control devices, where warning lights attached to these devices were thrown free, posing a potential threat to workers and other traffic (Bryden, 1990).

D. Design Element: Delineation of Crossovers/Alternate Travel Paths

Table 38. Cross-references of related entries for delineation of crossovers/alternative travel paths.

Applications in Standard Reference Manuals	
MUTCD Revision 3, Part VI (1993)	
Pg. 73, Sect. d Pg. 120, Item 5 Pg. 131, Fig. TA-12 Odd No. Pgs. 171-175, Fig(s). TA-32–TA-34 Odd No. Pgs. 183-195, Fig(s). TA-38–TA-44	Pg. 93, Sect. c, 3rd & 4th Bullets Pg. 121, Fig. TA-7 Pg. 155, Fig. TA-24 Pg. 179, Fig. TA-36

Studies have established that: (1) a substantial proportion of construction work-zone accidents occur in the taper and the crossover, where channelizing devices are usually located; (2) darkness is associated with an increase in the frequency of accidents in these areas; and (3) construction zones are associated with increases in the incidence of fixed-object, rear-end, and head-on accidents (Graham, Paulsen, and Glennon, 1977). Nemeth and Rathi (1983), studying accident types in construction zones on the Ohio Turnpike, found that 52.4 percent of the accidents were with fixed objects, and 68.3 percent of the crossover accidents involved collisions with channelizing devices or other objects. In this study, 69.4 percent of the accidents at the first curve of a crossover occurred at night. Nemeth and Migletz (1978) found that 60.7 percent of single-vehicle fixed-object accidents were collisions with drums and 29.8 percent of all accidents involved collisions with drums. They also found that the proportion of accidents involving construction objects (drums) at night is significantly higher than the proportion of daylight construction object accidents. The results of these studies highlight the need for highly conspicuous and properly installed and maintained channelizing devices.

The relationships between functional capabilities of older drivers and their performance that are likely to be of greatest operational significance as they approach and negotiate a crossover in a work zone can be summarized as follows. Age-related declines in acuity (both static and dynamic) and contrast sensitivity will delay recognition of channelizing devices and pavement markings and will delay comprehension of the information provided by advance warning signs. This information loss in the early stages of the driver's vehicle control task will be compounded by attentional and decisionmaking deficits shown to increase with increasing age, with age differences in performance magnified as serial processing demands for conflict avoidance and compliance with traffic control messages increase during the approach to the work zone. Age-related decrements in the useful field of view, selective attention, and divided attention/attention-switching capabilities will slow the initiation of a driver's response when a lane change is required prior to the transition zone, or maneuvering through channelization across the median. In addition, less efficient working memory processes may translate into riskier operations for older drivers in unfamiliar areas if concurrent search for and recognition of navigational cues is required; such demands disproportionately tax "spare capacity" for lanekeeping and conflict avoidance for older operators. Finally, the execution of vehicle-turning movements becomes more difficult for older drivers as bone and muscle mass decrease, joint flexibility is lost, and range of motion diminishes. Simple reaction time, while not significantly slower for older drivers responding to expected stimuli under nominal operating conditions, suffers operationally significant decrements with each additional response to an unexpected stimulus, e.g., as required in emergency situations. In addition, older drivers' increased sensitivity to glare and reduced dark adaptation ability will compound the difficulties described above while driving at night.

The National Transportation Safety Board (NTSB) has expressed concern about the lack of positive separation of opposing traffic in work zones (NTSB, 1992). The NTSB uses "positive barrier," or "positive separation of traffic," to refer to the use of concrete barriers to separate traffic, notably the Jersey-type barrier. (A number of States distinguish between these terms, using "positive separation" to describe various channelization treatments which do not necessarily involve use of a physical [Jersey] barrier.) The NTSB (1992) asserts that, *"Accident rates, particularly fatal accident rates, increase significantly when an interstate highway is switched from a four-lane, divided operation to a two-lane, two-way operation (TLTWO) during construction work."* Research bearing on the use of channelization and barrier delineation for TLTWO's is described below.

A crossover requires a change in direction and may require a reduction in speed. This requires adequate advance warning of the lane and speed reduction, conspicuous and unambiguous delineation/channelization in the transition zone, and conspicuous separation of opposing traffic the length of the TLTWO. One survey of drivers in Houston and Dallas (Texas) by Hawkins, Kacir, and Ogden (1992) found that only half of the respondents correctly understood that they should turn before reaching the CROSSOVER sign (D13-1) when this device was shown in a field placement in an arterial work zone. Of course, the D13-1 sign panel is identified in the MUTCD as a device used in permanent installations on divided highways, not as a temporary device for use in construction zones. The poor comprehension of motorists for such an explicit message is alarming, nevertheless, and suggests the need for heightened conspicuity of guidance information in this situation. Hawkins et al. recommended that the spacing of channelizing devices be decreased in the vicinity of a crossover to reduce drivers' confusion.

Next, Pang and Yu (1981) conducted a study to verify whether concrete barriers were justified at transition zones adjacent to TLTWO's on normally divided highways, based on accident experience in several construction zone TLTWO's. They found that 34 of the 44 total accidents that occurred in TLTWO's were within the transition zone. Four head-on accidents occurred on two-way, two-lane segments away from the transitions. The transition zone was defined as the roadway section at which traffic flow was converted from a four- to a two-lane operation and vice versa. The absence of opposing traffic precluded the occurrence of head-on accidents during the study period; however, more than one-half of the accidents (56 percent) had the potential of becoming head-on collisions. The authors concluded that on relatively low-volume highways, delineation devices appear to be adequate at transition zones, assuming they are placed properly. A regression analysis provided by Pang (1979) indicated that as annual average daily traffic increases, the accident rate at transition zones also increases, with a concurrent increase in the head-on accident rate at the transition zone.

Project duration and approach speed are two other variables that appear to affect the head-on accident rate at transitions (Pang and Yu, 1981). Graham (1977) concluded that as project duration increases, the accident rate at the transitions decreases. Expectancy issues were highlighted as a plausible explanation. Pang and Yu (1981) reported that because the accident rate in the transition zone increases with shorter project duration, concrete barriers may be necessary for short-term projects. However, long-term projects are expected to have a greater number of accidents owing to a longer period of exposure. Thus, installation of concrete barriers would be more economically justified for long-term projects than for short-term ones. With regard to approach speed, it can be expected that as speed to the transition increases, the chances of a head-on collision would also increase, due to the tendency of vehicles to stray out of their lanes at curves such as those present in transition zones. Pang and Yu (1981) suggested that concrete barriers appear to be justified at transition zones where approach speeds are high.

The conspicuity of concrete safety shaped barriers (CSSB's) is an important issue. Their composition provides little contrast with the roadway pavement, making them difficult to see at night, particularly in the rain, and under opposing headlight glare conditions. Proper barrier delineation treatments will provide drivers with a defined path during darkness and adverse weather conditions. Standard barrier delineation treatments include Type C steady-burn warning lights on top of the barrier, retroreflective devices on the top or side of the barrier, vertical panels placed on top of the temporary concrete barrier, and reflective pavement markings on the side of the barrier. The results of studies of barrier delineation in work zones have been mixed (Ullman and Dudek, 1988). For instance, Mullowney (1978) suggested that delineation should be mounted on the top of the barrier so it will retain its reflectivity longer and require less maintenance. However, Ogwoaba (1986) recommended side-mounted concrete barrier delineation so that the delineators are not masked by oncoming headlight glare. The size and brightness of delineators is another controversial topic, with some studies suggesting the use of larger but less bright devices (Davis, 1983; Bracket, Stuart, Woods, and Ross, 1984; Kahn, 1985) and others recommending smaller, brighter reflectors (Mullowney, 1978; Ogwoaba, 1986). Kahn (1985) found that the delineation of portable concrete barriers improved considerably through the use of cylindrical reflectors on top and smaller units on the side of the barrier at 7.6-m (25-ft) intervals. Delineator spacings ranging from 7.6 m to 61 m (25 ft to 200 ft) have been recommended by various studies.

Ullman and Dudek (1988) conducted a study of five barrier delineation treatments, using observations of driver performance to determine how different delineator types, spacings, and mounting positions on the barrier affect nighttime traffic operating in the travel lane next to the barrier. An additional objective of the study was to determine how the visibility and brightness of different types of delineators deteriorate over time because of dirt and road film; in a controlled field study, drivers ages 18–56 were asked to provide subjective evaluations of delineator brightness. The study was not conducted at a work zone, but was conducted on an illuminated urban freeway with four lanes in each direction. The CSSB was located 0.3 m (1 ft) from the inside travel lane. The five delineation treatments were: (1) top-mounted cube-corner lenses at 61-m (200-ft) spacing; (2) side-mounted cube-corner lenses at 15.2-m (50-ft) spacing; (3) top-mounted reflective brackets at 15.2-m (50-ft) spacing; (4) side-mounted reflective brackets at 61-m (200-ft) spacing; and (5) top-mounted reflective cylinders at 15.2-m (50-ft) spacing. The cube-corner reflector (treatments 1 and 2) had a diameter of 82.5 mm (3.25 in). The brackets (treatments 3 and 4) were 76 mm (3 in) high and 108 mm (4.25 in) wide, and were covered with high-intensity sheeting. The cylindrical tube (treatment 5) had a diameter of 76 mm (3 in) and was 152 mm (6 in) high, and was wrapped with high-intensity sheeting. Before-and-after data were obtained for the following measures of effectiveness: lane distribution, lane straddling, and lateral distance from the left rear tire to the bottom of the CSSB.

Results of the driver performance data collected by Ullman and Dudek (1988) showed that the treatments had very little practical effect on lane distribution. Lane-straddling rates at all of the treatment segments were low during the higher volume nighttime hours; however, a significant increase in lane straddling occurred for treatment 2. The data suggested that the combination of close delineator spacing and the side-mounted position may make some drivers too apprehensive of driving near the barrier. Lateral distance data showed significant differences during the higher volume nighttime hours for treatment 4 and treatment 5. Lateral distance distributions shifted away from the barrier at treatment 4 and closer to the barrier at treatment 5. Subjective evaluations for clean delineators showed brightness rankings to be the same for all treatments. Treatments 1-4 received adequate ratings from at least 80 percent of the subjects, while treatment 5 was rated adequate by only 50 percent of the subjects. With respect to each treatment's relative effectiveness in helping drivers maintain a safe travel path next to the CSSB, the rankings did not differ significantly; however, treatment 5 again received the worst score. Subjects stated that side-mounted delineators were preferable to top-mounted delineators because side-mounted delineation provided a more direct line of sight, a better indication of the location of the wall, and a more realistic perception of the lane width. For dirt-covered delineators, treatment 2 was rated as brightest and most effective, while treatment 5 was rated as dimmest and least effective. Although further research was deemed necessary due to limitations in the study scope and funding, a recommendation made by the study authors based on the delineators studied was to use cube-corner lenses for delineating CSSB's in narrow freeway median applications, because these delineators do not lose their reflectivity due to dirt and grime as quickly as those covered with high-intensity sheeting. In addition, for situations with limited lateral clearance, as is common with TLTWO, top-mounted delineation is recommended, because side-mounted close delineator spacing results in lane straddling if the barrier is located close to the travel lanes. Although subjects indicated a preference for close spacings, driver performance data did not show any differences between 15.2-m (50-ft) and 61-m (200-ft) spacing. The authors recommended that a 61-m (200-ft) spacing be considered maximum, and that closer spacings may be necessary for CSSB's on sharp curves. The recommendations were also deemed appropriate for CSSB's in work zones.

On divided highways with narrow medians, which are often created when barriers are used in crossover situations in work zones, drivers are subject to blinding glare from opposing vehicle headlights. This is particularly problematic for older drivers who have a reduction in their dark adaptation ability and increased sensitivity to glare. This results in reduced visibility of roadway alignment and channelization, and increases the possibility of accidents. Glare screens can solve the problem, as well as reduce rubbernecking and its associated problems. The Pennsylvania Department of Transportation discontinued the use of the standard glare-control mesh screen in 1976, based on maintenance difficulties, and has employed a paddle-type system in its place (Maurer, 1984). The system consists of plastic airfoil-shaped paddles, which when mounted resembles a picket fence. Results of a 5-year study have shown that the paddle-type system reduces headlight glare satisfactorily and is more cost-effective, both in terms of installation and maintenance, than metal mesh screen. The system was also found to be beneficial as a temporary control for channelizing traffic around a construction work zone, when screening was placed at the transition or the taper zone at the ends of the work zone (Maurer, 1984). Kelly and Bryden (1983) reported that a glare screen consisting of individual plastic louvers 900 mm (36 in) high, mounted vertically on a guiderail or median barrier spaced at 600-mm (24-in) centers, performed as expected in two safety improvement projects.

E. Design Element: Temporary Pavement Markings

Table 39. Cross-references of related entries for temporary pavement markings.

Applications in Standard Reference Manuals	
MUTCD Revision 3, Part VI (1993)	
Pg. 72, Sect. b Pg. 155, Fig. TA-24 Pg. 174, Item 4 Pg. 182, Item 5 Odd No. Pgs. 183-195, Fig(s). TA-38–TA-44	Pg. 131, Fig. TA-12 Odd No. Pgs. 171-175, Fig(s). TA-32–TA-34 Pg. 178, Item 7 Pg. 179, Fig. TA-36

Preconstruction centerlines and edgelines that are not obliterated may confuse drivers about the exact locations of lanes. The National Transportation Safety Board (1992) has reported that although guidelines exist for proper signing and striping in construction areas, the traffic control techniques used in many jurisdictions are not in compliance with the guidelines. Lewis (1985) stated that if drivers are presented with conflicting information (as may be the case in a work zone), they will generally choose to follow the pavement, as the pavement itself is a primary source of information for drivers. This points to a need for unambiguous pavement delineation patterns in work zones, to provide clear guidance—particularly at night and under adverse weather conditions—and to accommodate drivers with visual limitations resulting from age, fatigue, or alcohol consumption.

The research findings that have the greatest bearing on age differences in drivers' ability to acquire and use information provided by roadway delineation are a decline in spatial contrast sensitivity and acuity for older drivers, and a general slowing of response related to the specific deficit in visual search ability to rapidly discriminate more important from less important information in a driving scene.

Discrimination of the boundaries of the traveled way often involves only slight differences in the brightness of the road surface versus the shoulder or surrounding land. The ability to obtain such "edge information" depends upon a driver's sensitivity to contrast. Age differences in contrast sensitivity, beginning at approximately age 40 and becoming progressively more exaggerated with advancing age, demonstrate significant decrements in performance for older subjects (Owsley, Sekuler, and Siemsen, 1983). Under constant viewing conditions, older observers have lower contrast sensitivity especially in situations where there is a reduction in ambient light levels. A 60-year-old driver requires 2.5 times the contrast needed by a 23-year-old driver (Blackwell and Blackwell, 1971).

Age decrements in visual search and scanning capabilities are widely reported in gerontological research. Rackoff and Mourant (1979) measured visual search patterns for 10 young (ages 21–29) and 13 older (ages 60–70) subjects as they drove on a freeway under day and night conditions in low to moderate traffic. They reported that differences between young and older test subjects' performance were most apparent at night, and that older subjects required more time to acquire the minimum information needed for vehicle control. Thus, older drivers require delineation information that is optimal from the standpoints of both attention conspicuity and search conspicuity downstream, and that provides unambiguous path guidance cues for moment-to-moment steering control. Uncertainty about roadway heading and lane position has been cited by older driver focus group members as reasons for driving slower, for erratic maneuvers caused by last-second steering corrections, and for simply avoiding nighttime operations (Staplin, Lococo, and Sim, 1990). An exaggeration of the difficulties older drivers have in rapidly discerning the correct travel path may be expected in construction zones, where drivers must respond to temporary pavement markings that are often in competition with preexisting stripes and/or misleading informal cues provided by variation in the surface characteristics of the road, shoulder, or median.

These diminished capabilities must be considered in relation to specific information needs, when negotiating work zones, while also taking into account the time (distance) in which these needs must be satisfied. The information needs may be loosely contrasted according to the discrimination of continuous versus discrete roadway features, i.e., the perception and recognition of the boundaries of the traveled way, as opposed to a singular location which must be avoided (e.g., an island, barrier, or abutment) or where a path selection decision must be acted upon (e.g., a ramp gore, pavement width transition point, or intersection). Furthermore, delineation must provide information to a driver permitting roadway feature recognition both at "long" preview distances up to and sometimes exceeding 5 s travel time, and at the more immediate proximities (within 1 s travel time) where attention is directed for instant-to-instant vehicle control responses.

An investigation of age-related differences in the required contrast for pavement delineation showed that an older driver (ages 65-80) test sample required a level of contrast 20–30 percent higher than a young/middle-aged (ages 19–49) comparison group (Staplin et al., 1990). The differences became exaggerated with glare as an independent variable. An inevitable consequence of these age differences is an increased reliance on delineation elements for path guidance by older drivers under nighttime conditions, especially against oncoming glare. The "long preview" as well as the instant-to-instant steering control cues provided by pavement markings are critical to older drivers under these circumstances.

Raised pavement markers (RPM's) used for delineation of the centerline and edgelines in construction zones have been found to provide improved wet weather and nighttime reflectivity, and are particularly useful when lanes are diverted from their original path (Spencer, 1978). Davis (1983) reported that, compared with paint, day-night/wet-night visible RPM's improved construction zone traffic performance significantly. In this study, the markers were associated with decreased lane-change frequency and night lane encroachments. In before-and-after comparisons of accident frequencies in two construction projects, the number of accidents and fatalities decreased as a function of RPM installation (Niessner, 1978). In a study investigating vehicle guidance through work zones, Shepard (1989) recommended that closely spaced RPM's should be used as a supplement to existing pavement striping in areas where the roadway alignment changes.

Dudek, Huchingson, and Woods (1986) conducted a study on a test track to examine the effectiveness of temporary pavement markings for use in work zones. Ten candidate treatments were tested during the day, and the most effective treatments were examined at night. All treatments were tested only under dry weather/dry road conditions. The candidate treatments are presented in table 40 and included patterns with stripes, RPM's, and combinations of stripes and RPM's. Treatment 1 was the control condition in the study.

Table 40. Temporary pavement marking treatments evaluated by Dudek, Huchingson, and Woods (1986).

Treatment	Description
1*	4-ft stripes (4 in wide) with 36-ft gaps (control condition)
2*	2-ft stripes (4 in wide) with 38-ft gaps
3*	8-ft stripes (4 in wide) with 32-ft gaps
4*	2-ft stripes (4 in wide) with 18-ft gaps
5*	Four nonreflective RPM's at 3-1/3-ft intervals with 30-ft gaps and one retroreflective marker centered in alternate gaps at 80-ft intervals
6*	Three nonretroreflective and one retroreflective RPM at 3-1/3-ft intervals with 30-ft gaps
7	2-ft stripes (4 in wide) with 48-ft gaps
8	Treatment 2 plus RPM's at 80-ft intervals
9*	Two nonretroreflective RPM's at 4-ft intervals with 36-ft gaps plus one retroreflective RPM centered in each 36-ft gap
10	1-ft stripes (4 in wide) with 19-ft gaps

* Treatments evaluated both day and night 1 ft = 0.305 m

Results of both daylight and nighttime testing indicated that there were no practical differences between treatments when comparing measures of effectiveness developed from speed and distance measurements. Practical differences were arbitrarily defined as at least 6.5 km/h (4 mi/h) for speed measures and 0.3 m (1 ft) for distance measures. The greatest number of erratic maneuvers during daylight occurred for treatments 7 and 8, which consisted of 0.6-m (2-ft) stripes and long gaps. Drivers referred to 0.6-m (2-ft) stripes as dots. The subjective data indicated that treatments 5, 6, and 9 were preferred, under both daylight and nighttime conditions. Reasons given were that RPM's clearly identify curves, are highly visible at a great distance, provide noise and vibration when drivers cross them, and stand out more than tape markings. Of the treatments without RPM's, treatment 3 was the drivers' choice, for both lighting conditions, while treatment 2 was rated as least effective.

Because subjects tend to perform best when in a proving ground setting and because the setting is not always sensitive enough to discern small differences between candidate treatments, Dudek, Huchingson, Creasey, and Pendleton (1988) conducted field studies to compare the safety and operational effectiveness of 0.3-m (1-ft), 0.6-m (2-ft), and 1.2-m (4-ft) temporary broken line pavement markings on 12.2-m (40-ft) centers in work zones. The study was conducted at night on rural two-lane, two-way highways with 2.0-degree horizontal curvatures, level to rolling terrain, and average speeds between 80.5 km/h (50 mi/h) and 99.8 km/h (62 mi/h). In terms of speed, lateral distance, encroachment, erratic maneuver, and speed profile data for the sample of vehicles with headways of 4 s or more, there were no differences in driver performance between the 0.3-m (1-ft), 0.6-m (2-ft), and 1.2-m (4-ft) striping patterns. Analysis of subjective evaluations of the effectiveness of the markings found that the 0.3-m (1-ft) stripe was generally rated as poorest, but its mean ranking was not significantly different from that of the 0.6-m (2-ft) and 1.2-m (4-ft) stripes. Drivers generally preferred the longer stripes, but there was no evidence that the 0.6-m (2-ft) or 1.2-m (4-ft) stripes were superior to the 0.3-m (1-ft) stripe. In a discussion of the conditions present during this research, Ward (1988) stated that all sites had 3.7-m (12-ft) lanes with 1.2-m (4-ft) to 3-m (10-ft) shoulders, the marking material was highly retroreflective yellow tape laid over very black new pavement overlays, and there were no edgelines; therefore, the drivers' focus was a "brilliant ribbon of yellow to follow," resulting in no difference in driver performance between the three stripe lengths. Most important was that none of the treatments were judged as extremely effective, although the 0.3-m (1-ft) stripe was rated as poorest, and there was a slight preference for the 1.2-m (4-ft) lengths. This is consistent with results obtained by Dudek et al. (1986), where subjects rated 2.4-m (8-ft) stripes with 9.8-m (32-ft) gaps as the best striping treatment (when RPM's were not available). In the Dudek et al. (1986) study, drivers preferred the treatments with longer stripes, shorter gaps, and RPM's. Hence, the results of the Dudek et al. (1988) study may be applicable only to pavement overlay projects on two-lane, two-way rural roadways, and may not translate to other highway work-zone situations.

Harkey, Mera, and Byington (1992) conducted a study to determine the effects of short-term (interim) pavement markings on driver performance under day, night, wet, and dry weather conditions. The three marking patterns tested included: (1) 0.6-m (2-ft) stripes with 11.6-m (38-ft) gaps and no edgelines; (2) 1.2-m (4-ft) stripes with 11-m (36-ft) gaps and no edgelines; and (3) 3-m (10-ft) stripes with 9.2-m (30-ft) gaps and edgelines. The measures of effectiveness included lateral placement of the vehicle on the roadway, vehicle speed, number of edgeline and lane line encroachments, and number of erratic maneuvers (e.g., sudden speed or directional changes and brake applications). For each operational measure, the 3-m (10-ft) markings resulted in better driver performance than either the 0.6-m (2-ft) or 1.2-m (4-ft) temporary marking patterns. Drivers traveled 1.2 km/h (0.76 mi/h) slower on segments with 1.2-m (4-ft) markings and 3.3 km/h (2.02 mi/h) slower on segments marked with 0.6-m (2-ft) markings than on segments marked with 3-m (10-ft) stripes and edgelines. In addition, compared with the 3-m (10-ft) pattern, drivers encroached over the lane or edgeline 66 percent more in the presence of the 1.2-m (4-ft) temporary marking and 139 percent more in the presence of the 0.6-m (2-ft) markings. These values increased dramatically under night and wet weather conditions. Comparisons of driver performance between the 1.2-m (4-ft) and 0.6-m (2-ft) markings showed the following: (1) the speed at which drivers traveled decreased as the length of the lane line decreased; (2) drivers positioned their vehicles closer to the center of the lane as the length of the line increased; (3) the variability of vehicle placement within the lane increased as the length of the lane line decreased; (4) the number of encroachments increased as the length of the lane line decreased; and (5) all operational measures were negatively affected by adverse weather conditions. Results provided evidence of significant decreases in driver performance associated with both the 0.6-m (2-ft) and the 1.2-m (4-ft) markings, but drivers performed better with the 1.2-m (4-ft) stripes compared to the 0.6-m (2-ft) stripes. The results suggested that while it may not be practical to place full markings (3-m [10-ft]) segments with 9.2-m [30-ft] gaps as specified by MUTCD part 3A-6) on a temporary basis, measures should be taken to prevent reductions in driver performance which result in increased accident potential; such measures include the use of longer temporary markings and the appropriate use of warning signs to indicate a change in the pavement marking pattern.

GLOSSARY

AAAFTS. American Automobile Association Foundation for Traffic Safety.

AASHTO. American Association of State and Highway Transportation Officials.

Ambient conditions. The visual background or surrounding atmospheric and visibility conditions.

Angular motion sensitivity. The ability of an observer to detect changes in the apparent distance and direction of movement of an object as a function of the change in the angular size of the visual stimulus on the observer's retina.

Angular velocity threshold. The rate of change in angular size of a visual stimulus that is necessary for an observer to discern that an object's motion has increased or decreased.

Annual average daily traffic (AADT). The total volume passing a point or segment of a highway facility in both directions for 1 year divided by the number of days in the year.

ATSSA. American Traffic Safety Services Association.

Attraction signing. Information/supplemental signs featuring logos or verbal messages pointing out places to visit or food, gas, and rest stop locations.

Barnes Dance timing. Type of exclusive signal timing phase where pedestrians may also cross diagonally in addition to crossing either street. Also referred to as scramble timing.

Brake reaction time. The interval between the instant that the driver recognizes the presence of an object or hazard on the roadway ahead and the instant that the driver actually applies the brakes.

Buttonhook ramp. J-shaped ramp that connects to a parallel or diagonal street or frontage road, which is often well removed from the interchange structure and other ramps.

Changeable message sign (CMS). Sometimes called portable changeable or variable message sign. This traffic control device has the flexibility to display a variety of messages to fit the needs of the traffic and highway situation.

Channelization. The separation or regulation of conflicting traffic movement into definite paths of travel by the use of pavement markings, raised islands, or other suitable means, to facilitate the safe and orderly movement of both vehicles and pedestrians.

Chevron signs. A chevron symbol (sideways "V") in black, against standard yellow background, on a vertical rectangle. Used as an alternate or supplement to standard delineators and to large arrow signs.

CIE. Commission Internationale de l'Éclairage (International Commission on Street/Highway Lighting).

Cloverleaf interchange. A form of interchange that provides indirect right-turn movements in all four quadrants by means of loops. Generally used where the turning and weaving volumes are relatively low. This type of interchange eliminates all crossing conflicts found in a diamond interchange but requires more area. The cloverleaf type of interchange can have one or two points of entry and exit on each through roadway.

Complete interchange lighting (CIL). Includes lighting in the interchange area on both the acceleration and deceleration areas plus the ramps through the terminus.

Composite photometry. Light measurement applied to a high-mast lighting system that employs a counterbeam arrangement, to take advantage of the efficiency with which pavement luminance can be increased with light directed upstream, while enhancing positive contrast through additivity of vehicle headlighting with the light directed downstream.

Concrete safety shaped barrier (CSSB). Commonly used median barrier where there is heavy vehicle travel and narrow medians.

Contrast. *See* **luminance contrast.**

Contrast sensitivity. Ability to perceive a lightness or brightness difference between two areas. Frequently measured for a range of target patterns differing in value along some dimension such as pattern element size and portrayed graphically in a contrast sensitivity function in which the reciprocal of contrast threshold is plotted against pattern spatial frequency or against visual angle subtended at the eye by pattern elements (such as bars).

Critical gap. The gap (distance to nearest vehicle) in oncoming or cross traffic that a driver will accept to initiate a turning or crossing maneuver 50 percent of the time it is presented, typically measured in seconds.

Dark adaptation. Adjustment of the eye to low levels of illumination, which results in increased sensitivity to light.

Decision sight distance (DSD). The distance required for a driver to detect an unexpected or otherwise difficult-to-perceive information source or hazard in a roadway environment that may be visually cluttered, recognize the hazard or its threat potential, select an appropriate speed and path, and initiate and complete the required safety maneuver safely and efficiently.

Depth perception. The ability to distinguish the relative distance of objects in visual space, used to interpret their motion over multiple observations.

Diamond interchange. The simplest and perhaps most common type of interchange. This type of interchange contains a one-way diagonal-type ramp in one or more of the quadrants. The diamond interchange provides for all movements to and from the intersecting road.

Diverge steering zone. Used with interchange/ramp exit models, it is the distance upstream from the exit gore at which a driver begins to diverge from the freeway.

Divided attention. The ability of a driver to allocate attention among tasks or stimuli in the roadway environment, where more than one task or stimulus is perceived to be important to safe performance at a given time.

Divided highway. Roadway that is separated by a median.

Dynamic visual acuity. Acuteness or clarity of vision for an object that has angular movement relative to the observer. Acuity depends on sharpness of retinal focus, sensitivity of nervous elements, oculomotor coordination, interpretative faculty of the brain, and contextual variables.

Edgeline visibility. The detection/recognition of painted pavement surface delineation along roadway edges.

Exit gore area. The area located immediately between the left edge of a ramp pavement and the right edge of the mainline roadway pavement at a merge or diverge area.

FARS. Fatal Accident Reporting System.

FHWA. Federal Highway Administration.

Full diamond interchange. Interchange with a one-way diagonal-type ramp in each quadrant.

Gap acceptance. The decision by a driver that there is sufficient time and/or distance ahead of an approaching vehicle to allow safe performance of a desired crossing or merging maneuver.

Gap judgments. The judgment of a driver of the time and/or distance ahead of an approaching vehicle traveling in a lane that the driver wishes to turn across or merge into.

Gap search and acceptance (GSA) zone. Used with interchange/ramp entry models, it is the zone in which the driver searches, evaluates, and accepts or rejects the available lags or gaps in the traffic stream for execution of a merging maneuver.

Guard (guide) rail. Protective barrier along a roadway to prevent vehicles from leaving the roadway.

Half-diamond interchange. An interchange with a one-way diagonal-type ramp in two adjacent quadrants. This type of interchange is appropriate to situations in which traffic demand is predominantly in one direction.

High-mast lighting. Illumination of a large area by means of a cluster of luminaires which are designed to be mounted in fixed orientation at the top of a high mast (generally 25 m [80 ft] or higher).

High-spatial-frequency stimulus. A visual target characterized by fine detail.

Horizontal alignment. The linear (tangent) character or specific degree of curvature describing the geometry of a defined section of highway in plain view.

IIHS. Insurance Institute for Highway Safety.

Illuminance. The density of luminous flux (rate of emission of luminous energy flow of a light source in all directions) incident on a surface; the quotient of the flux divided by the area of the surface, when the surface is uniformly illuminated.

Illumination. The amount of light falling onto a surface.

Initial acceleration (IA) zone. Used with interchange/ramp entry models, it is the zone in which the driver accelerates to reduce the speed differential between the ramp vehicle and the freeway vehicles to an acceptable level for completing the merge process.

In-service brightness level (ISBL). The brightness level of a delineation treatment at an intermediate point in its anticipated service life; this value varies by type of delineator, type of wear (traffic level), and environmental conditions.

Interchange (grade separation). A system of interconnecting roadways that provides for the movement of traffic between two or more highways on different levels.

Intersecting angle (skew). The angle formed by the intersection of two roadways (other than a 90-degree angle).

Intersection (at grade). The general area where two or more highways join or cross without grade separation, including the roadway and roadside facilities for traffic movements within it.

Intersection sight distance (ISD). The unobstructed view of an entire (at-grade) intersection and sufficient lengths of the intersecting highway to permit control of the vehicle to avoid collisions during through and turning movements.

ISTEA. Intermodal Surface Transportation Efficiency Act.

ITE. Institute of Transportation Engineers.

Joint flexibility. An aspect of the physical condition of the driver that can be assessed to determine whether the driver has sufficient strength to turn the steering wheel, apply the brakes, and generally control the vehicle.

Legibility Index (LI). Used to describe the relative legibility of different letter styles, it is calculated from the distance at which a character, word, or message is legible divided by the size of the letters on the sign.

Limited sight distance. A restricted preview of the traveled way downstream due to a crest vertical curve or horizontal curvature of the roadway, or to blockage or obstruction by a natural or manmade roadway feature or by (an)other vehicle(s).

Luminaire. A complete lighting unit consisting of a lamp or lamps together with the parts designed to distribute the light, to position and protect the lamps, and to connect the lamps to the power supply.

Luminance. The luminous intensity or brightness of any surface in a given direction, per unit of projected area of the surface as viewed from that direction, independent of viewing distance. The SI unit is the candela per square meter.

Luminance contrast. The difference between the luminance of a target area and a surrounding background area, divided by the background luminance alone (e.g., lane marking minus lane pavement surface, divided by pavement surface.)

Measures of effectiveness (MOEs'). Descriptions of driver or traffic behavior which quantify the level of safety or the quality of service provided by a facility or treatment to drivers, passengers, or pedestrians; examples include vehicle speed, trajectory, delay, and similar measures, especially accidents, plus indices of performance such as reaction time. In research studies, the MOE's are the dependent measures (e.g., the effects/behaviors resulting from introduction of a treatment or countermeasure).

Median barriers. A longitudinal system of physical barriers used to prevent an errant vehicle from crossing the portion of a divided highway separating traffic moving in opposite directions.

Merge steering control (MSC) zone. Used with interchange/ramp entry models, it is the zone in which the driver enters the freeway and positions the vehicle in the nearest lane on the mainline.

Minimum required visibility distance (MRVD). The distance necessary to permit detection and comprehension, plus driver decisionmaking, response selection, and completion of a vehicle maneuver, if necessary.

MUTCD. *Manual on Uniform Traffic Control Devices for Streets and Highways.*

NCHRP. National Cooperative Highway Research Panel.

Negative offset. A term used to describe the alignment of opposing left-turn lanes at an intersection; this geometry exists when the left boundary of one left-turn lane, when extended across the intersection, falls to the right of the right boundary of the opposite left-turn lane.

NHTSA. National Highway Traffic Safety Administration.

No turn on red (NTOR). This message on signs is used to indicate that a right turn on red (or left turn on red for one-way streets) is not permitted at an intersection.

NTSB. National Transportation Safety Board.

Ocular media. The internal structure of the eye, including the aqueous, through which light entering through the cornea must be transmitted before reaching the photosensitive retina.

Ocular transmittance. The amount of light reaching the retina relative to the amount incident upon the cornea.

Osteoarthritis. A degenerative form of arthritis.

Parclo loop ramp. A (partial cloverleaf) interchange with loops in advance of the minor road with direction of travel on the freeway; and in the same interchange area, an interchange with loops beyond the minor road.

Partial interchange lighting (PIL). Lighting on an interchange that consists of a few luminaires located in the general areas where entrance and exit ramps connect with the through traffic lanes of a freeway (between the entry gore and the end of the acceleration ramp or exit gore and the beginning of the deceleration ramp).

Peak intensity. The maximum strength of a traffic signal maintained through a defined viewing angle; measured in candelas.

Pedestrian control device. A special type of device (including pedestrian signal indications and sign panels) intended for the exclusive purpose of controlling pedestrian traffic in crosswalks.

Pedestrian crosswalk. An extension of a sidewalk across an intersection or across a roadway at a midblock location to accommodate pedestrian movement.

Perception-reaction time (PRT). The interval between a driver's detection of a target stimulus or event and the initiation of a vehicle control movement in response to the stimulus or event.

Positive offset. A term used to describe the alignment of opposing left-turn lanes at an intersection; this geometry exists when the left boundary of one left-turn lane, when extended across the intersection, falls to the left of the right boundary of the opposite left-turn lane.

Post-mounted delineators (PMD's). Retroreflective devices located serially at the side of a roadway to indicate alignment. Each delineator consists of a flat reflecting surface, typically a vertical rectangle, mounted on a supporting post.

Raised pavement markers (RPM's). Used as positioning guides and/or as supplements or substitutes for other types of markings, these markers conform to the color of the marking for which they serve as a positioning guide, can be mono- or bi-directional, and are fastened into the pavement with the reflector surface visible above the road surface.

Reaction time (RT). The time from the onset of a stimulus to the beginning of a driver's (or pedestrian's) response to the stimulus, by a simple movement of a limb or other body part.

Rheumatoid arthritis. A usually chronic disease of unknown cause characterized by pain, stiffness, inflammation, swelling, and sometimes destruction of joints. Drivers with this condition sometimes require compensatory equipment for their vehicle. In acute conditions, individuals should not drive because of weakness and extreme tenderness in the joints of the wrists and hands.

Right turn on red (RTOR). Unless otherwise specified by traffic signal control signage, this practice permits a driver to proceed with a right turn on a red signal after stopping at signalized intersections. It provides increased capacity and operational efficiency at a low cost.

Route Marker Reassurance Assembly. Consists of a cardinal direction marker (i.e., east, west, north, and south) and a route marker.

Saccadic movement. A change in visual fixation from one point to another by means of a quick, abrupt movement of the eye.

Scissors off-ramp. A condition where one-way traffic streams cross by merging and diverging maneuvers onto exit ramps. Drivers tend to go straight ahead onto an off-ramp instead of turning left.

Selective attention. The ability, on an ongoing moment-to-moment basis while driving, to identify and allocate attention to the most relevant information, especially embedded when within a visually complex scene and in the presence of a number of distractors.

Senile miosis. An aging characteristic involving an excessive smallness or contraction of the pupil of the eye.

Sight distance. The length of highway visible to the driver.

Sight triangle. In plan view, the area defined by the point of intersection of two roadways, and by the driver's line of sight from the point of approach along one leg of the intersection, to the farthest unobstructed location on another leg of the intersection.

Situational awareness. The selective attention to and perception of environmental elements within a specified space and time envelope, the comprehension of their meaning, and the projection of their status in the near future.

Slip ramp. A diagonal ramp, more properly called a cross connection, which connects with a parallel frontage road.

Small target visibility (STV). A proposed criterion for roadway lighting. The concept assumes that increased target visibility results in both increased nighttime safety and improved nighttime driver performance, a surrogate for reduced accident risk.

Speed-change lane (SCL). Used in interchange/ramp exit models, it refers to the speed-change maneuver on deceleration lanes segmented components.

Steering control (SC) zone. Used with interchange/ramp entry models, it is the zone where positioning of the vehicle along a path from the controlling ramp curvature onto the speed-change lane is accomplished.

Stereopsis. Binocular visual perception of three-dimensional space based on retinal disparity.

Stopping sight distance (SSD). The sight distance required to permit drivers to see an obstacle soon enough to stop for it under a defined set of reasonable worst-case conditions, without the performance of any avoidance maneuver or change in travel path; the calculation of SSD depends upon speed, gradient, road surface and tire conditions, and assumptions about the perception-reaction time of the driver.

Temporary pavement marking treatment. This treatment primarily involves the application of paint or tape striping and has been shown to be important for effective vehicle guidance at highway work sites.

T-intersection. An intersection that involves three legs, where one leg is perpendicular to the other two legs. There are several types of this intersection, such as plain, with turning lanes, and channelized.

Traffic control device (TCD). The prime, and often the only, means of communicating with the driving public. These devices (e.g., signs, markings, signals, islands) must be used discriminately, uniformly, and effectively to ensure correct driver interpretation and response.

Transient adaptation factor. A reduction in target contrast caused by the process of transient visual adaptation.

Transient visual adaptation (TVA). The process in which the (driver's) eye fixates upon roadway locations or surrounding environments at different luminance levels, continuously adapting to higher and lower levels; this process temporarily reduces contrast sensitivity.

TRB. Transportation Research Board.

Trumpet interchange. A three-leg interchange where a connecting highway terminates and where only a small amount of traffic moves between the terminating highway and one of the two legs of the freeway. The trumpet is laid out so that this minor traffic moves via a 200-degree loop.

Two-quadrant cloverleaf interchange. A type of partial cloverleaf where most traffic leaving one highway turns to the same leg of the intersecting highway.

Useful field of view (UFOV). That area surrounding the point of fixation within which one can perform more complex tasks. This might include discriminating among letters or geometric figures, identifying a target against a complicated background display, or combining a secondary task in the periphery with an ongoing task in the forward (central) field of view.

Variable message sign (VMS). *See* **changeable message sign.**

Veiling glare. Stray light entering the eye that reduces the contrast of a target upon which the driver has fixated; this may result from the driver's direct view of light sources, such as opposing headlights or roadway luminaires, or from light reflected from surfaces near the target's location.

Vertical curve. The parabolic curve connecting the two approach grades on either side of a hill.

Visual acuity. The ability of an observer to resolve fine pattern detail. Acuity is usually specified in terms of decimal acuity, defined as the reciprocal of the smallest resolvable pattern detail in minutes of arc of visual angle. "Normal" or average acuity is considered to be 1.0 (a resolution of 1-min arc).

Visual clear (VC) zone. Used with interchange/ramp entry models, this refers to the zone that provides a buffer between the driver and the end of the acceleration lane, where the driver can either merge onto the freeway in a forced maneuver or abort the merge and begin to decelerate at a reasonable rate.

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